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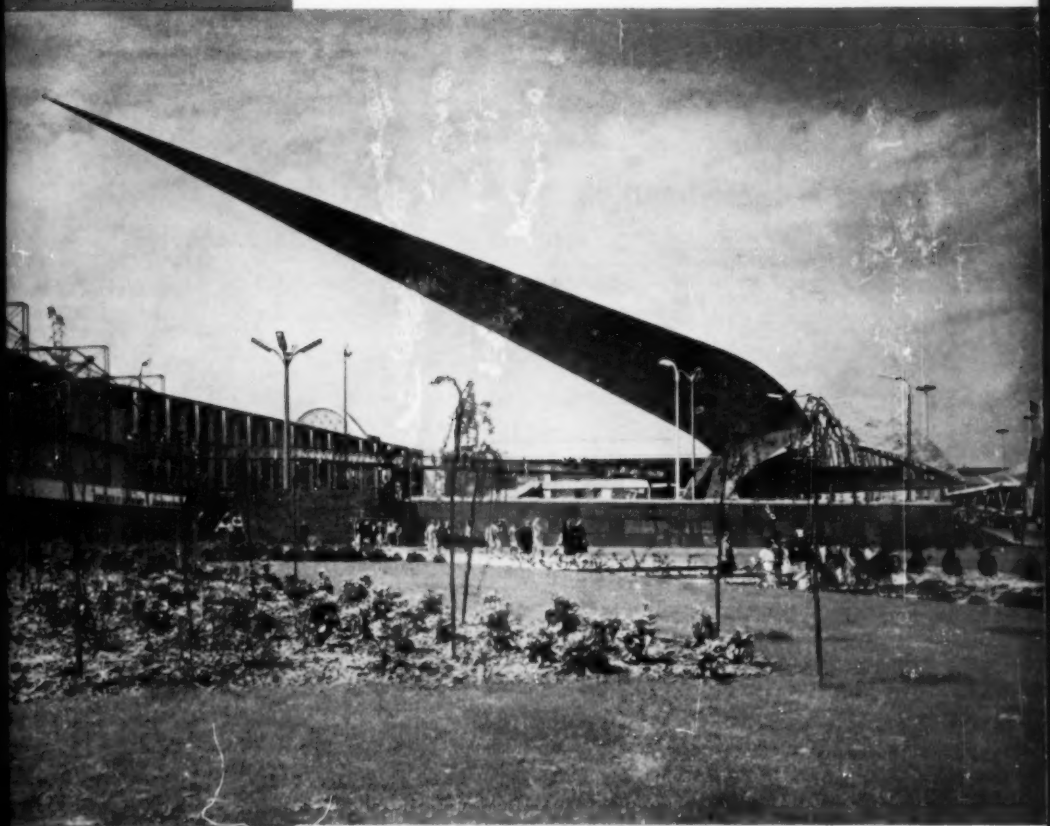

ACI
PROCEEDINGS
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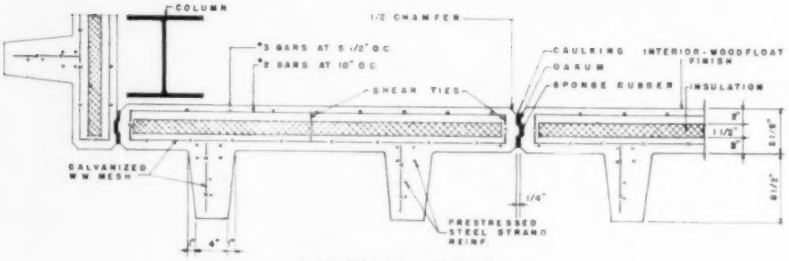
Reports on work of Comité Européen du Béton (p. 1)
and Reinforced Concrete Research Council (p. 73)

Stressing Design

Prestressed concrete double tees standing on end provide imaginative new wall treatment



HORIZONTAL SECTION

Glass fibre insulation was sandwiched between layers of concrete in the crossbar of the double tee unit to provide an excellent thermal barrier.

● Double tee beams of prestressed concrete, best known for their value in roof and floor systems, are now finding an attractive new use. A recently completed switching station of the Public Service Electric & Gas Company of Newark, N. J. has two walls made of side-by-side 10-ton vertical double tees running the full height of the building —nearly 61 feet—and creating an interesting pattern.

A third wall, featured in the photograph above, is topped by tees 39½ feet tall. The bottom section of this wall employs removable flat concrete slabs, to facilitate the replacement of major equipment.

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Coming next month in the JOURNAL

Five papers presented at the 1960 annual convention at the symposium sponsored by ACI Committee 201, Durability of Concrete in Service, will be published in the August issue.

LEWIS H. TUTHILL reviews "Conventional Methods of Repairing Concrete," discussing factors affecting bond, permeability, cracking, durability, and appearance.

EARL J. FELT suggests methods for cleaning and preparing the old surface for grouting, and for placing concrete, in his paper "Repair of Concrete Pavements."

"Prepakt Method of Concrete Repair" is described as employed both in the restoration of old concrete and masonry structures and in certain types of new construction by RAYMOND E. DAVIS.

BAILEY TREMPER discusses the "Repair of Damaged Concrete with Epoxy Resins."

Experience with "Pneumatically Applied Mortar for Restoring Concrete Structures" is related by O. N. KULBERG.

J. TAUB and A. M. NEVILLE offer Part 1 of a five-part series on "The Resistance to Shear of Reinforced Concrete Beams. Part 1—Beams Without Web Reinforcement."

On the Cover—The Civil Engineering Pavilion at the 1958 International Exhibition at Brussels. For details on design and construction see the article by A. Paduart and J. Van Doosselaere on p. 51 of this month's Journal.—Photo courtesy Ministere des Travaux Publics et de la Reconstruction (Belgium)

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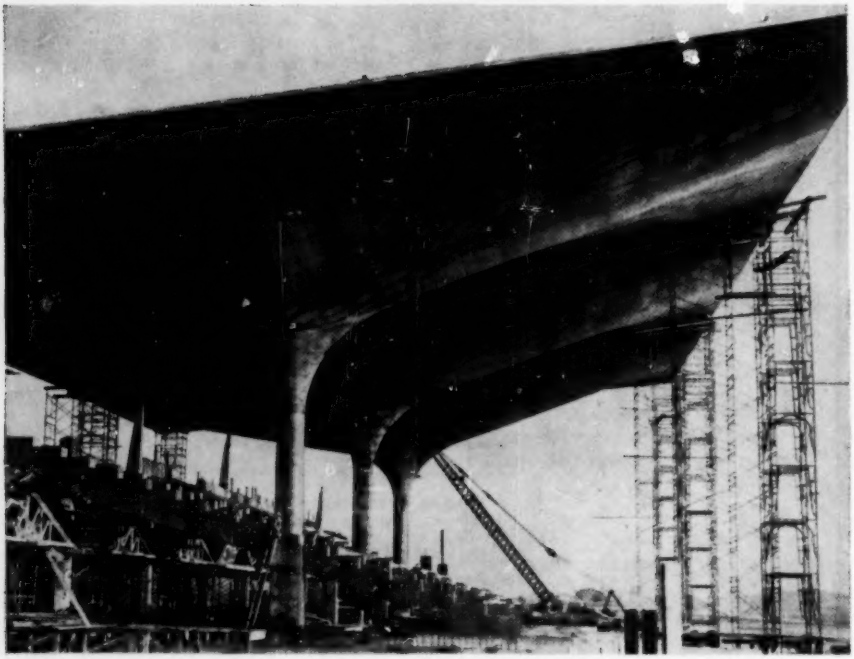

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Title No. 57-1

Researches Toward a General Flexural Theory for Structural Concrete

By HUBERT RÜSCH

This paper is directed toward formulation of a general flexural theory based on a careful study of all important factors regarding the properties of concrete. The fact that strength and deformation of concrete depend on time is considered. The theory is based on recent tests permitting determination of the behavior of the compression zone in flexure for continuous load increase at different strain rates, and for constant sustained load. Having derived stress-strain relationships for these various types of loading, other factors were studied systematically, such as effect of concrete strength, position of neutral axis, and shape of cross section. The general theory developed is primarily a study of the true behavior of structural members. Since simplified assumptions are avoided, it naturally does not lead to simple formulas such as are desired for structural design. The theory fulfills the important function of furnishing a reliable method for the evaluation of simplified design formulas. It is also possible, however, to present all new concepts and results of this theory in the form of a simple diagram which can be used for the solution of design problems for selected cross sections ranging from pure bending to pure compression, regardless of concrete quality and the type of steel used, and independent of whether prestressing is applied or not.

■ RESEARCH IN THE STRUCTURAL CONCRETE FIELD is faced today with problems of unusual challenge. We find ourselves in a period of change characterized by the abandonment of the elastic theory in favor of the plastic theory, and by a conversion from allowable stresses as a basis of design to ultimate strength design. Although these trends have persisted for some time, the new methods are finding slow acceptance among design engineers in some countries. This is probably at least in part due to the fact that structural engineering can look back on a thousand-year tradition, and this tradition is by its nature a conservative one. Another reason of equal importance is the lack of detailed and extensive knowledge regarding the properties of materials desirable in the development and introduction of new methods.

In recent decades, progress has been made toward replacing structural design methods disregarding plastic properties of materials by

ACI member **Hubert Rusch** has been, since 1945, a professor and director of the Engineering Materials Laboratory, Technical University, Munich, Germany. Dr. Rusch has won much prominence in Europe and South America through his design and construction of outstanding reinforced concrete structures. He has played an active part in reinforced concrete research, and in the development of shell structures (for which he received the Longstreth Medal prior to World War II), prestressed concrete, and precast construction.

new ones, which represent actual conditions to a greater degree. Numerous investigations of structural concrete have been conducted leading to several new design theories which generally are in good agreement with test results in the case of pure flexure. However, these theories start from very different, sometimes even contradictory, assumptions about the physical behavior of the component materials. This is probably a major reason why none of the new methods has found world-wide acceptance. An engineer seeks to analyze the true behavior of structures. He cannot be convinced by approximately correct results obtained on the basis of widely different assumptions. The agreement between the results of various theories in design, however, is not at all surprising since only the case of under-reinforced beams has often been cited in comparisons with test data. The tensile force in the steel at failure is determined entirely by the yield point; the lever arm of the internal forces is insensitive to assumptions regarding concrete stress. Only tests of over-reinforced members can furnish a true measure of the validity of a flexural theory. There is a need for a theory which is not restricted to approximate results in a limited range. Such a theory must be based on the actual properties of the materials and must be valid for all cases of loading, from pure bending to pure compression.

The reason why authors differ so widely in appraising the physical behavior of concrete in flexure probably lies in the fact that their knowledge is based almost entirely on beam tests. Only three conditions are available for the evaluation of such tests: the equilibrium condition, the deformation condition, and Bernouilli's assumption of plane sections remaining plane. As the number of unknowns is generally greater than the number of equations, some plausible assumptions must be made in the evaluation of certain quantities. As the required quantities are closely interrelated, it is quite understandable that one may thus arrive at widely different solutions.

It is only lately that attempts were made to establish the needed relationships in a direct manner. First among these should be mentioned tests on centrally and eccentrically loaded prisms conducted by Hognestad,[1,2] Moenaert,[3] and Rüsch,[4] which led to an extensive clarification of the behavior of the compression zone in flexure under *short-time* load only.

However, strength decreases under the action of *sustained loads.* Creep of the concrete leads to an increase in concrete strain in the extreme compressive fiber. This results in a lower neutral axis and a reduction of the lever arm of the internal forces. Consequently, the stress in the reinforcement becomes higher. A more important consideration is the reduction of concrete strength under the action of sustained load. This problem has been studied in detail only in recent years.

The following discussion reports results of new tests whose objective was to study effects of time such as age of concrete and duration of loading. These tests constitute the basis of the new flexural theory.

Notation

Notation is defined in Fig. 1 and frequently also in the text. In addition, some frequently used symbols are:

A_c = area of concrete compression zone (for symmetrical bending of a rectangular cross section, $A_c = bc$)

α = stress block factor = $C/A_c f_c' = f_{avg}/f_c'$

$c_{(a,t)}$, c_1, c_λ = coefficients defined by Eq. (2)

f_{avg} = average stress in concrete compression zone at ultimate strength

f_c = concrete stress

f_c' = concrete cylinder strength

f'_{c28} = 28-day cylinder strength

$f'_{c(a,t)}$ = strength of concrete failing under sustained load at the time t days after loading at an age of a days

f_{su} = stress in tensile steel at ultimate strength of reinforced concrete member

f'_{su} = stress in compression steel at ultimate strength of reinforced concrete member

k_u = ratio c/d at ultimate strength, Fig. 1

j_u = value of j at ultimate strength, Fig. 1

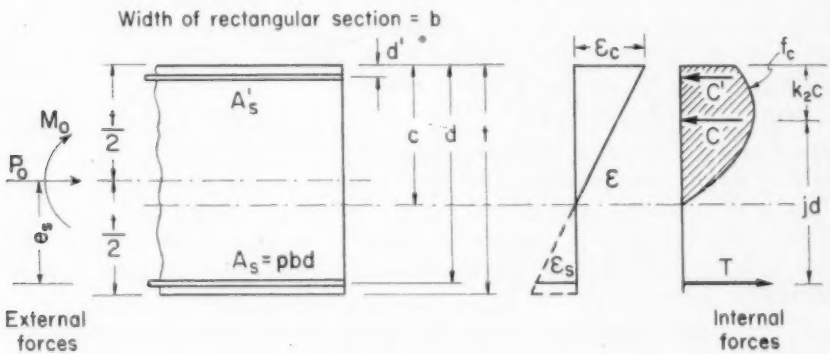

Fig. 1—Notation

M = moment of internal concrete force about centroid of tensile reinforcement; at ultimate strength $M = M_u$

m = relative internal resisting moment of concrete compression zone defined by Eq. (1), at ultimate strength $m = m_u$

M_o = moment of external forces with respect to centroid of cross section

P_o = external axial force acting in the centroid of cross section

e_s = distance between centroid of tensile reinforcement and centroid of cross-section (for symmetric loading of rectangular section $e_s = d - t/2$)

ε = concrete strain

ε_o = concrete strain in extreme fiber

ε_u = concrete strain in extreme fiber at ultimate strength

ε_s = tensile strain in reinforcement

ε_{su} = tensile strain in reinforcement at ultimate strength

RATE OF LOADING EFFECTS

Standards of some countries require that in tests of materials the load be applied at a certain constant rate. However, this requirement cannot be satisfied at high loads for materials exhibiting an elasto-plastic behavior. For example, in testing steel in the yield range, the rate of deformation would become extremely high. Even if our testing machines could satisfy this requirement, such testing would still have to be ruled out because it leads to completely misleading results.

The above-mentioned requirement has another disadvantage. Under constant rate of loading, the stress-strain diagram can be recorded only up to a maximum stress, after which further load increase is no longer possible. In this study, we wish to examine the portion of the stress-strain curve beyond maximum stress, since it has a considerable effect on the stresses produced in a concrete structure.

For these reasons, the requirement contained in some standards specifying a constant rate of loading must be modified for research purposes. It should be replaced by a more rational requirement, namely that all tests of materials be carried out under constant rate of strain. One can then determine the descending portion of the stress-strain curve as the deformation continues to increase further under decreasing load after the maximum stress is reached.

With increasing duration of loading, as mentioned earlier, strength drops and deformation increases. Hence, the magnitude of the selected rate of strain has a strong effect on the shape of the stress-strain curve. This was specially pointed out by Rasch[5] who studied this effect on three different quality classes of concrete at strain rates from 0.001 per min to 0.001 per 70 days under concentric load in the Munich Materials Laboratory.

In carrying out these tests the compressive force was regulated manually in such a way that strain increased at the desired constant rate. Due to the heterogeneous nature of concrete, strain under a

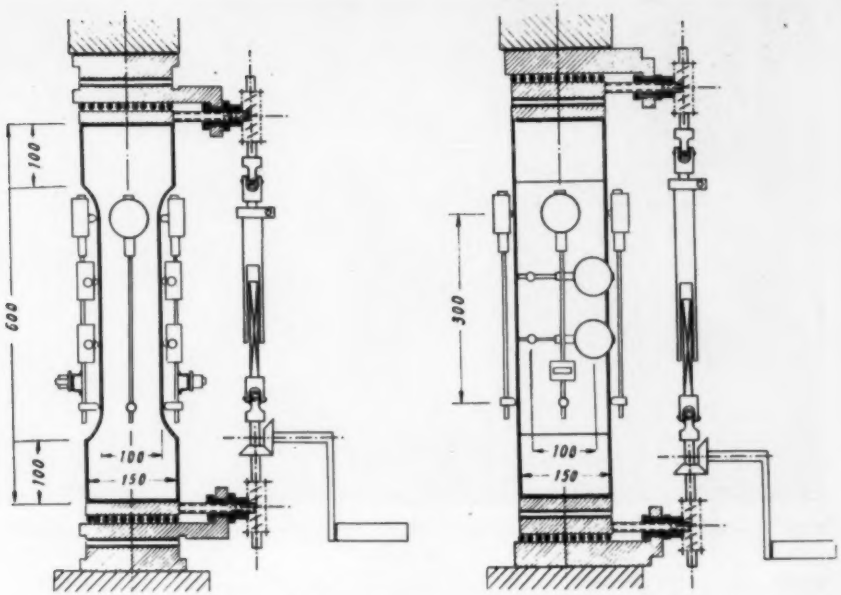

Fig. 2—Test arrangement and centering device (dimensions in millimeters). Left—Front view. Right—Side view

concentrically applied load is not necessarily the same on all sides of the test specimen. To eliminate this effect, the centering device shown in Fig. 2 was built into the testing machine to permit lateral displacement of the loaded test specimen in two directions with respect to the force axis. At all load levels, the position of the specimen was so adjusted by lateral displacements that strain on all four sides of the specimen at midheight remained equal. Such manual regulation becomes difficult at very fast rates of strain, and is too time consuming at very slow rates. This difficulty is eliminated by a testing machine, developed in the Munich laboratory, provided with electronically programmed controls and automatic recorders as shown in Fig. 3. Conventional testing machines are built in such a way that one has to apply a given load to the test specimen, and record the corresponding deformation. With the new machine, however, one can subject the specimen to a predetermined deformation, and record the corresponding load. The built-in programmed control makes it possible to increase the deformation at a constant rate. The machine then records automatically the desired stress-strain diagram.

Fig. 4 shows examples of the results obtained by the described method for concretes of a 3000-psi average strength and loaded 56 days after casting. The deformations shown in the diagram are not purely

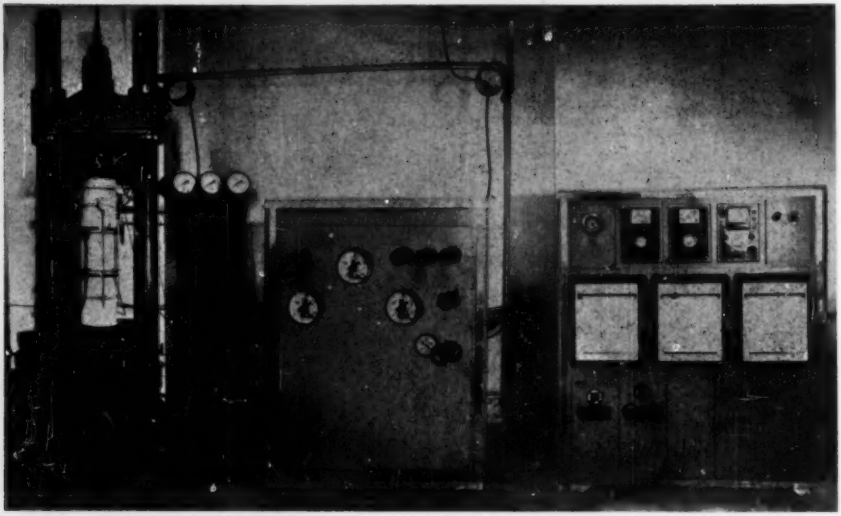

Fig. 3—Testing machine with electronically programmed control and automatic recorders

elastoplastic. The slower the rate of loading, the greater are the effects of creep and shrinkage. Naturally, there is a series of secondary factors in addition to those of strength and time, such as type of cement and cement content, grading and modulus of elasticity of aggregates, temperature, and moisture, which influence the stress-strain curve. Hence,

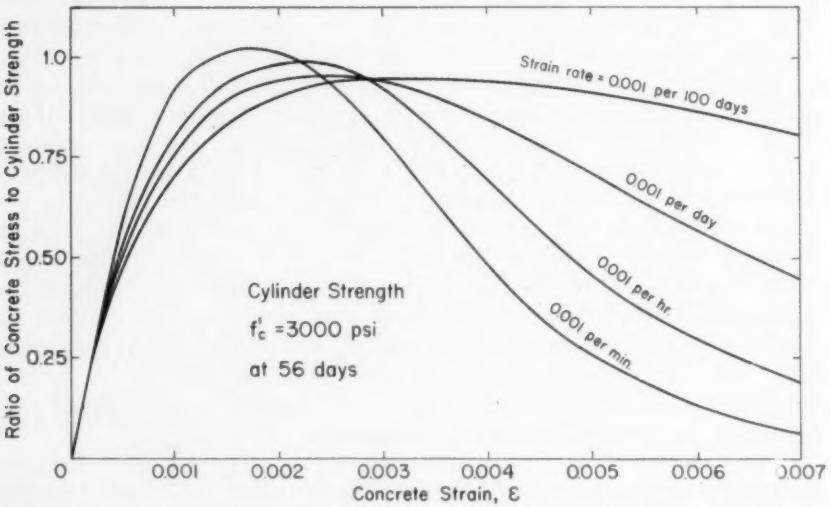

Fig. 4—Stress-strain curves for various strain rates of concentric loading

curves such as those shown in Fig. 4 may vary within certain limits. However, they will always follow the trend indicated by this diagram, which characterizes a predominating influence of time.

STRESS-STRAIN RELATIONSHIP IN FLEXURE

Seeking to establish the distribution of stresses in the concrete compression zone in flexure, it should first be considered that every "fiber" in this zone undergoes strain at a different rate. Assuming that cross sections remain plane, the rate of strain becomes proportional to the distance from the neutral axis. Furthermore, the desired stress distribution depends on the nature of load; for increasing load, on the rate at which the load is increased; for constant load, on the duration of loading.

In his paper,[5] Rasch proposes that the stress distribution in the compression zone in flexure be derived from stress-strain curves obtained from concentrically loaded prisms. How this can be done is illustrated schematically below for an example of load increasing at constant rate of strain. The stress-strain curves in Fig. 4 are used as a basis.

Fig. 5 shows schematically the derivation of the stress-strain relationship in the concrete compression zone in flexure. It is determined by the requirement that the strain of every fiber in the flexural compression zone is attained in the same interval of time, 1 hr in the chosen example. The stress corresponding to strain of 0.001 must then be selected from that stress-strain curve in Fig. 4 which corresponds to a deformation rate of 0.001 per hr. Similarly, stresses for strain of 0.003 and 0.005 are obtained from the stress-strain curves which were deter-

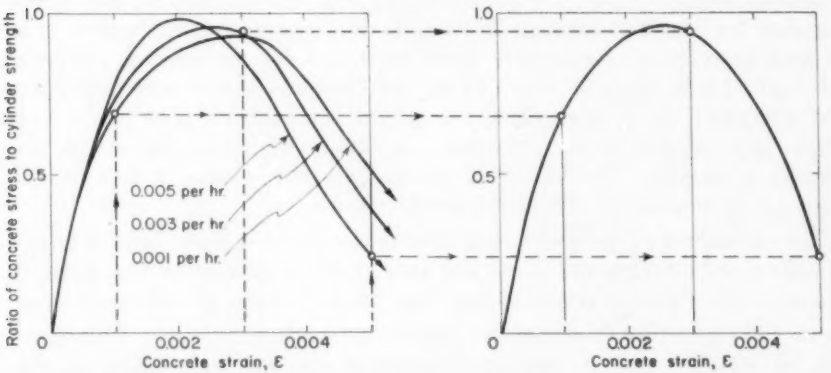

Fig. 5—Determination of stress-strain relationship in flexure (schematic only): (left) Stress-strain curves for concentric compression and various strain rates; (right) Stress-strain relationship for eccentric compression after 1 hr of loading at constant strain rates

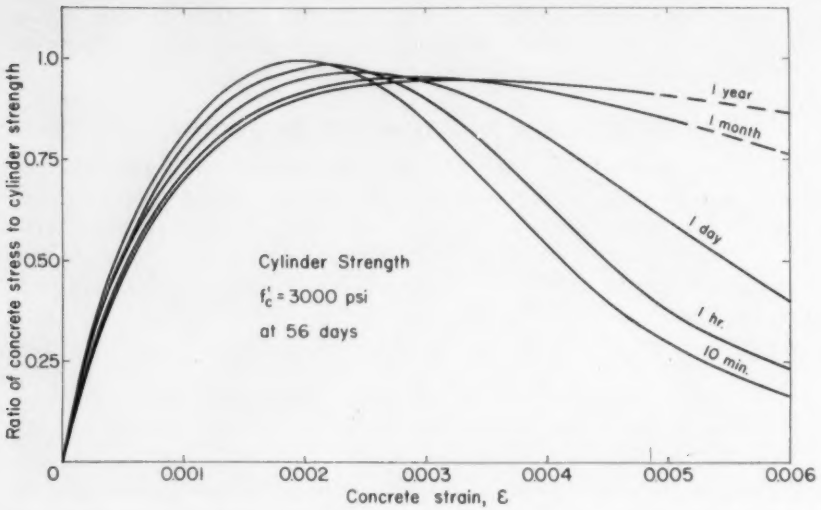

Fig. 6—Stress-strain relationships for eccentric compression after various durations of loading at constant strain rates

mined at strain rates of 0.003 and 0.005 per hr, respectively. When the stress values determined in this manner are plotted against the corresponding strains, the stress distribution for the compression zone in flexure is obtained. This applies for all loading conditions attained in 1 hr in the various fibers, under constant yet different rates of strain. The magnitude of the strain in the extreme fibers is determined by the given magnitude of load.

Several relationships are presented in Fig. 6, obtained in the above manner for various loading durations. All curves apply to the case of load increasing at constant strain rate and for an average concrete strength of 3000 psi at 56 days. These curves show clearly how important the effect of time is on the behavior of the compression zone in flexure. With decreasing rate of straining, the value of maximum stress decreases gradually. The effect of creep, however, causes a rise of the descending branch of the stress-strain curves.

The described proposal for the determination of a basic law of stress distribution pre-supposes that, for each rate of strain in the various fibers of the flexural compression zone, there appear the same stresses as in corresponding fibers of a concentrically loaded prism. This need not be strictly true in reality, because a mutual interaction of the variously deformed fibers of the compression zone in flexure may be possible due to transverse deformation. In any case the resulting errors are small — as proven by comparative tests — and do not sub-

stantially change any derived conclusions. They can be neglected in favor of a systematic study of the time factor.

COMPRESSIVE STRAIN IN EXTREME FIBERS

An important question must be answered at this stage: for an individual case, what are the values of compressive strain in the extreme fibers according to the new laws of stress distribution? This question has a simple answer: In every load test the strain in the extreme fibers is always that which will yield the required internal moment. The ultimate load is that corresponding to the maximum attainable value of the internal moment.

Fig. 7 demonstrates how the stress law can be analyzed with this aspect in mind. For a definite cross section of the compression zone in flexure and a chosen position of the neutral axis (Fig. 7 concerns a rectangular cross section and $c/d = 0.4$), the resisting moment is plotted as a function of the strain in the extreme fibers. To determine the magnitude of the internal moment, the stress block factor α and the coefficient k_2 are used to compute the magnitude and position of the concrete compressive force. These coefficients can be derived for any assumed value ε_c of the strain in extreme fibers from the stress distributions shown in Fig. 6. The curve of the relative internal

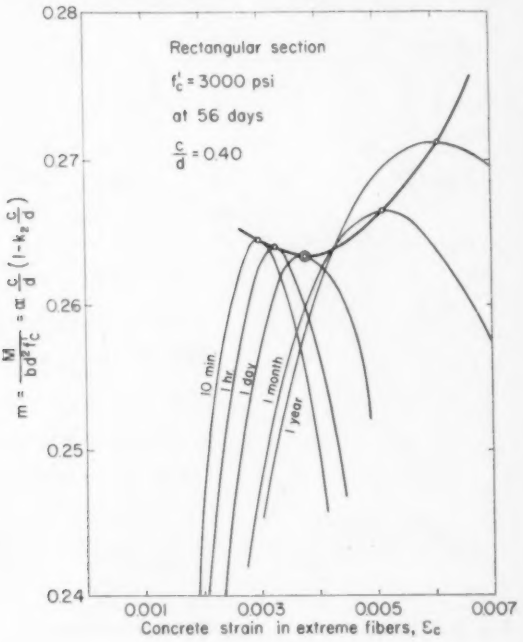

Fig. 7—Resisting moment as a function of strain and time

moment of the compression zone about the centroid of the tensile steel,

$$m = \frac{M}{bd^2 f'_c} = \alpha \frac{c}{d}\left(1 - k_3 \frac{c}{d}\right) \quad \text{.....(1)}$$

shows a clearly defined maximum in Fig. 7 for each loading duration which maximum corresponds to the ultimate moment, M_u. The corresponding strain in the extreme fibers is the ultimate strain, ε_u.

Fig. 7 further shows that the stress distributions applying to different loading durations lead to different values of ultimate moment and ultimate strain. Joining the various ultimate moments by a curve shown as a heavy line in Fig. 7, the dependence of the ultimate moment on duration of loading is seen. This curve usually shows a clear minimum. Hence, assuming that load is increased in such a manner that the rates of strain in the various fibers remain constant with time, there exists a definite duration of loading which leads to the lowest ultimate moment.

EFFECT OF POSITION OF THE NEUTRAL AXIS AND OF SHAPE OF CROSS SECTION ON ULTIMATE STRAIN

The results shown in Fig. 7 apply to one chosen position of the neutral axis only and to a rectangular cross section. When the same computations are carried out for different positions of the neutral axis and for various shapes of cross section, indications are obtained regarding

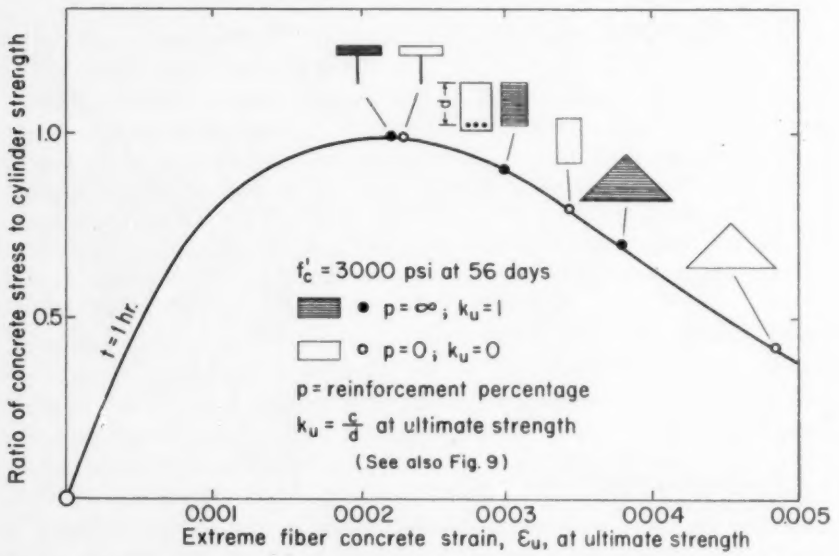

Fig. 8—Ultimate strain as a function of cross section and position of neutral axis

the degree to which the extreme fiber strain and the stress distribution at ultimate strength depend on these variables.

Examples of the results of such investigations are shown in Fig. 8. It shows extreme fiber strains at ultimate strength for several typical cross sections after 1 hr of loading and for an average concrete strength of 3000 psi at 56 days. The two mathematically extreme cases of position of the neutral axis were considered. The solid circles in the figure represent the case where the neutral axis is located at the centroid of the tension steel, the open circles denote the case when it lies at the upper edge of the cross section. In actual cases involving bending of reinforced concrete beams, the neutral axis will be between these two extreme positions. Fig. 8 shows clearly that the shape of the cross section has a decisive effect on the value of ultimate strain. For a triangular compression zone, a case which often occurs in biaxial flexure of columns, the ultimate strain is twice that for a T-beam. When the neutral axis is located at the centroid of the tension steel, this can be

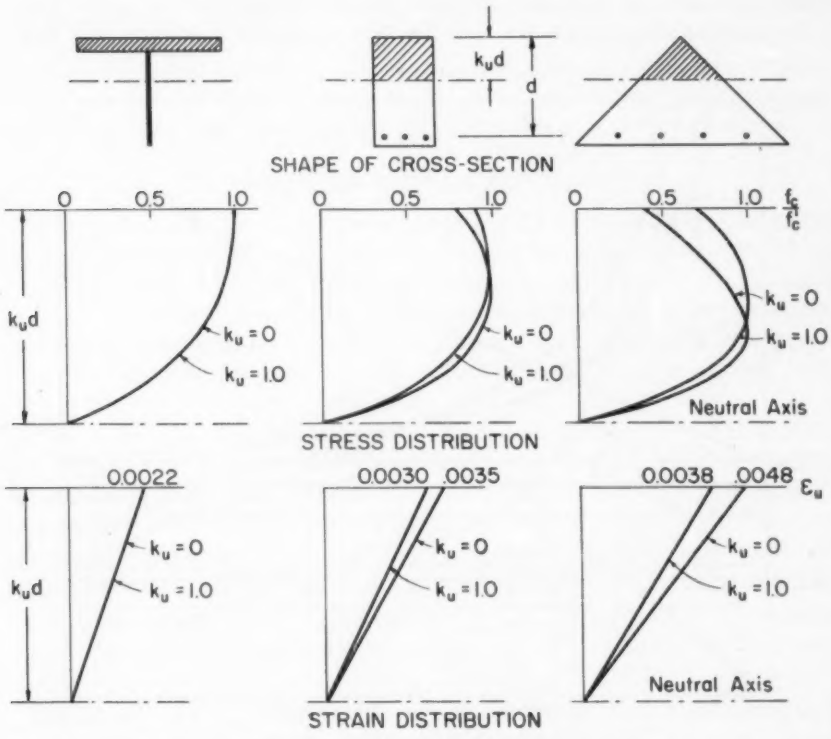

Fig. 9—Strain and stress distribution at ultimate strength after 1 hr, $f_c' = 3000$ psi at 56 days

qualitatively understood by the stress distribution shown in Fig. 9. For the idealized T cross section, most of the compression zone area is located in the upper flange, so that maximum internal moment occurs for that extreme fiber strain which gives maximum stress in the flange. For the triangular cross section, however, a major portion of the compression zone is located closer to the tension steel. Hence, maximum internal moment occurs for a relatively large extreme fiber strain giving maximum stress at some distance below the apex of the triangle. This theoretical deduction was confirmed by tests.

Fig. 8 also shows that the position of the neutral axis is of marked influence. This effect is least for T-beams and greatest for a triangular compression zone, which can be understood as follows. In an idealized T cross section, the magnitude of the lever arm of the internal forces is almost independent of the position of the neutral axis. Thus, at ultimate load, the strain in the extreme fibers is always close to that which yields the maximum internal compressive force. Conditions are different for a triangular compression zone. The magnitude of the ultimate moment is strongly affected by the length of the lever arm of the internal forces as well as by the magnitude of the internal

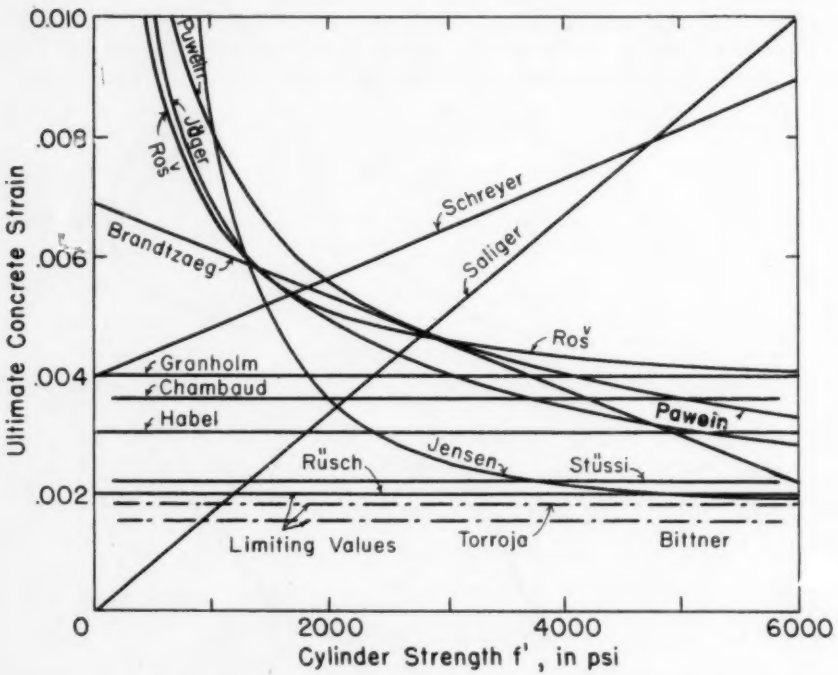

Fig. 10—Ultimate strain-concrete strength relationships

compressive force. In a rectangular cross section the position of the neutral axis still has an appreciable effect on ultimate strain.

The values reproduced in Fig. 8 and 9 apply to average concrete strength of 3000 psi and a rate of deformation at which failure takes place in 1 hr. In spite of this limitation, the value of the ultimate strain in the extreme fiber varies from about 0.0022 to 0.0048. If other concrete strengths and other rates of deformation are considered, even more pronounced differences may be expected.

A very important point is arrived at here. Most ultimate strength design theories advanced heretofore started out from the assumption that the stress distribution in the flexural compression zone, as well as the value of ultimate strain, were constant or at most dependent on concrete strength. In reality, however, these important design quantities are affected not only by concrete strength, but even to a greater degree by the *rate of loading*, the *position of the neutral axis*, and the *shape of the cross section*.

This qualitatively explains the wide divergence of the values of ultimate strain arrived at on the basis of earlier theories published by various authors, as shown in Fig. 10. It can even be argued that all these reported values, though apparently contradictory, can actually occur side-by-side. They were probably recorded under widely different conditions, and the fundamental error consisted in generalization.

EFFECT OF SUSTAINED LOADS

This discussion has so far dealt with a loading method characterized by a constant rate of strain. Even at very slow strain rates, therefore, maximum load exists only for a very short time period. This loading method is close to conditions existing in laboratory tests of structural specimens. The loading of actual structures generally takes place in a more unfavorable manner. In such structures, the load is applied relatively quickly and is then held constant.

The difference between these two types of loading is schematically illustrated in Fig. 11. The curves drawn with dashed lines correspond to loadings at constant strain rates. The rates shown correspond to failure after 1 hr, 1 day, and 3 months. The curves drawn by full lines correspond to loads applied in about 20 min and then held constant. It is seen that, for failure after a given period of time, the constant loads lead to somewhat lower failure loads than loading at constant strain rates. The investigations described below contribute to the study of these phenomena.

For some years the Munich Materials Laboratory has conducted tests of the effect of sustained load on the strength and deformation of concrete.[6,7,8] Difficulties in keeping a relatively high load constant over

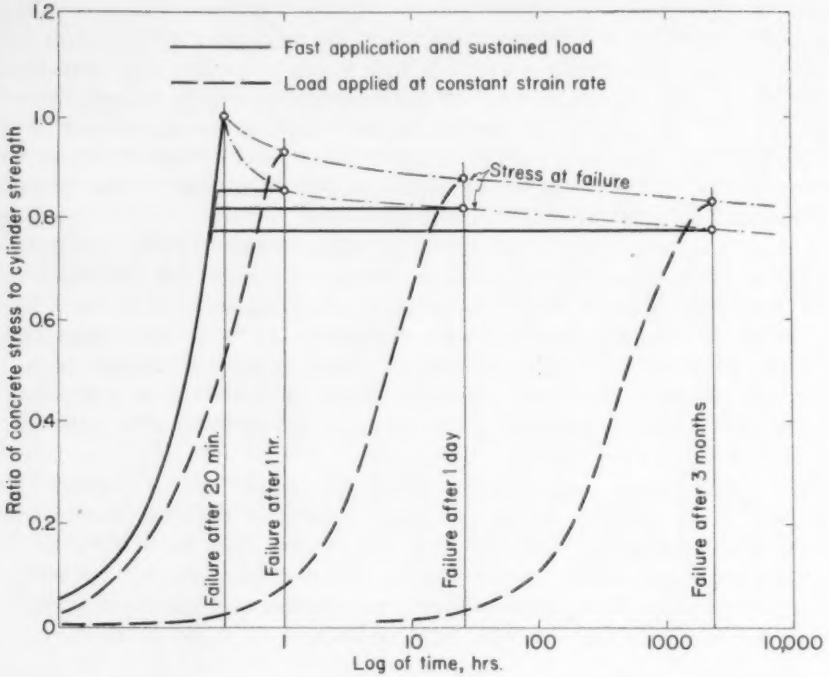

Fig. 11—Influence of type of loading on ultimate strength (schematic behavior of concentrically loaded concrete prisms)

a long period of time, even when concrete contracts under creep, were overcome by the development of a hydraulic loading arrangement such as is shown diagrammatically in Fig. 12. The piston rests on a rubber capsule which is connected to a constant pressure source. Piston leakage is eliminated, and the load can be kept constant practically without attendance. Eccentricity of loading can be varied by the hand crank arrangement shown, thus displacing the specimen laterally with respect to the force axis. Deformation was measured by mechanical strain gages.

A climate-controlled testing laboratory was used, in which a number of test specimens were subjected to load simultaneously. In these tests several identical specimens were subjected to concentric sustained load. The magnitude of load was varied for individual specimens. Its ratio to the ultimate load in a short-time test was designated as degree of loading. In this manner, the load can be determined at which the test specimen will just be able to sustain over an infinite length of time without breaking. The corresponding average compressive stress is called the sustained load strength. In addition, one observed increases in

deformation, i.e., creep under very high degrees of loading. Fig. 13 and 14 show selected results of these investigations for concentric loading and concrete of a 5000 psi average strength. The eccentricity of load was varied in other tests series.

Fig. 13 shows the influence of the degree of loading on the deformation and time elapsed up to failure. All specimens were concentrically loaded 56 days after casting. The failure in a conventional short-time test occurs in about 20 min at an ultimate strain of about 0.0025. For specimens subjected to a sustained load with degree of loading less than one, two families of curves with different characteristics are obtained. As long as the degree of loading is higher than that corresponding to the sustained load strength, the deformations eventually increase rapidly and lead to failure. For loading below the sustained load strength, the deformation curves become stabilized and approach limiting values of strain.

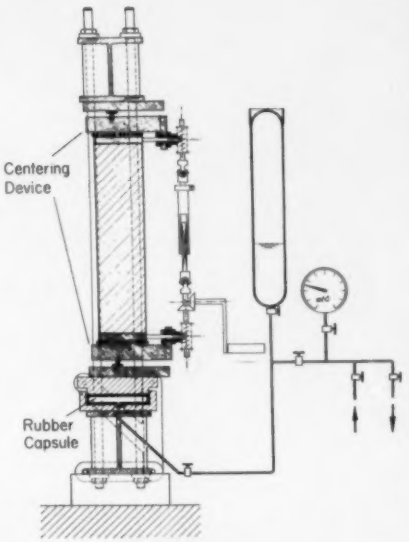

Fig. 12—Testing arrangement for sustained load tests

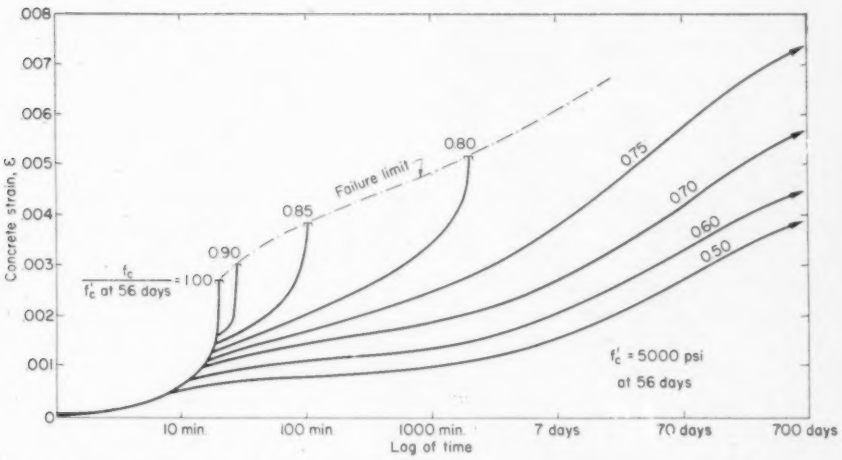

Fig. 13—Strains under sustained load applied at concrete age 56 days (concentric loading of prisms at 56 days)

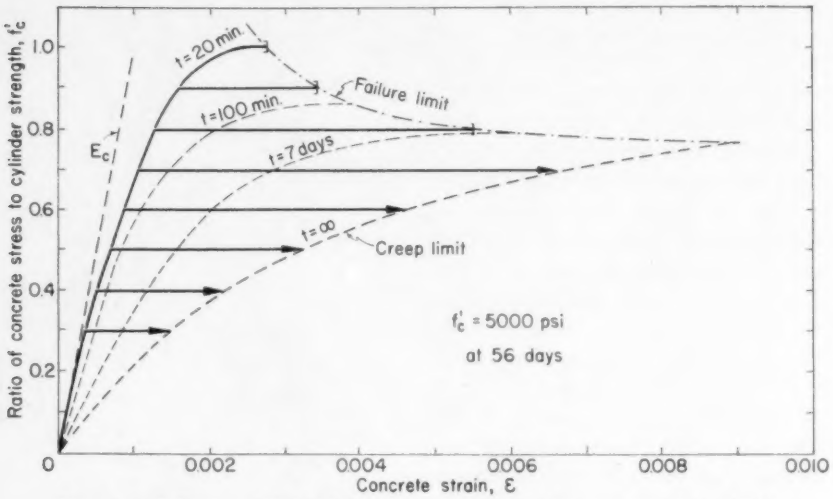

Fig. 14—Influence of load intensity and duration on concrete strain

The same results are given in Fig. 14 in terms of curves for applied load versus strain with time after loading at an age of 56 days as a parameter. It can be expected that the limiting line on the left of the diagram will become close to a straight line for extremely short durations of loading, until finally it will coincide with the elastic straight line relationship denoted E_c in the figure. On the lower right the diagram is limited by the creep deformations corresponding to an infinite duration of loading. At the top, the diagram is limited by the failure line, which shows decreasing strength for increasing load duration. The two parameter curves shown between these limits correspond to conditions after 100 min and after 7 days, respectively. The limiting lines of the diagram described above, enclose all possible relationships between stress and strain.

The Munich tests have shown that sustained load strength of a concentrically loaded concrete specimen amounts to at least 75 percent, and on the average to about 80 percent of the strength determined in a short-time test. The short-time strength is then defined as the strength of an identically cast and identically old specimen, which remains without load and is tested in a standard short-time test of about 10 min duration at the time when the twin specimen under sustained load has collapsed. This short-time strength depends on age and storage conditions and is usually greater than the strength at the time of loading which is used as a basis in Fig. 13 and 14.

The ratio of sustained-load strength to short-time strength is, according to our evidence, independent of concrete strength. In accordance with the chosen definitions it is also fairly independent of concrete age at

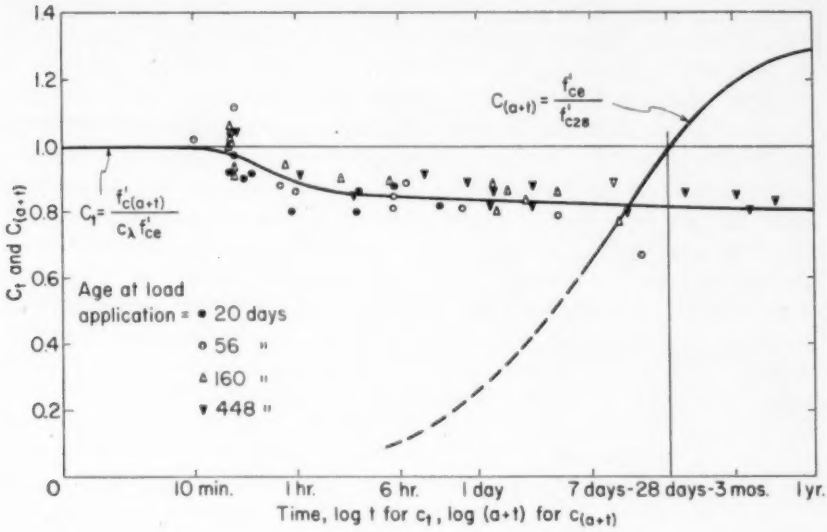

Fig. 15—Effects of time on strength

application of sustained load. However, since short-time strength increases with time after casting, the time elapsed until failure still has a pronounced effect on the absolute value of the sustained-load strength.

Concrete loaded shortly after casting is subjected to two different effects. The strength reduction caused by sustained load is counteracted by the strength increase with time. As the strength reduction due to sustained loading is particularly pronounced immediately following application of load, failure of a young concrete constitutes a danger only during the first days after load application. The effect of additional hardening becomes predominant thereafter. In contrast to this, for a very old concrete which has reached practically its full strength before it is loaded, failure under sustained load may occur after very long periods of loading. This effect of age at loading can be approximately expressed by the following relation, set up on the basis of the test findings shown in Fig. 15 and 16. The strength of a concrete failing under sustained load at the

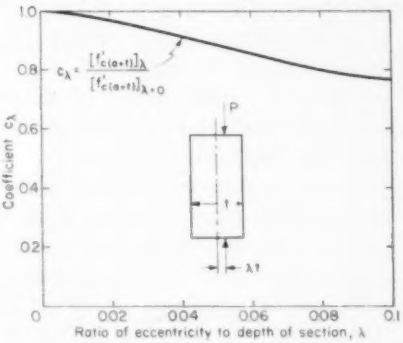

Fig. 16—Effect of eccentricity on strength

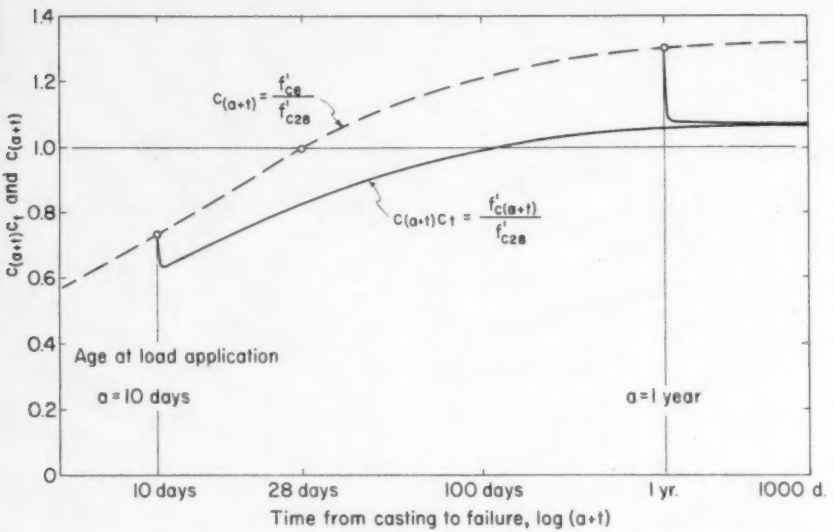

Fig. 17—Effect of age at load application on sustained load strength

time t days after loading at an age of a days is:

$$f'_{c(a+t)} = f'_{c28}\, c_{(a+t)}\, c_t\, c_\lambda = f'_{ca}\, c_t\, c_\lambda \quad \ldots\ldots (2)$$

in which f'_{c28} represents the concrete strength at the age of 28 days; a denotes the age of concrete at the beginning of sustained load application; t is duration of load after the full load (applied in a period of 20 min) has been reached; f'_{ca} is the short-time concrete strength at an age of $(a + t)$ days; and λ is the relative eccentricity of the load $\lambda = e/t$. In this manner, the effects of continued hardening and of eccentricity of load are expressed by the coefficients $c_{(a+t)}$ and c_λ, which can be obtained directly from experimental data. The values of the coefficient c_t, expressing the influence of the sustained load, were derived by means of Eq. (2) from sustained load test data. The curve to the right in Fig. 15 shows the effect, $c_{(a+t)}$, of continued curing on short-time strength for a concrete with a 28-day strength of 4300 psi. The flat curve beginning to the left expresses the effect of sustained loading, c_t, and was determined for sustained loadings applied from 20 to 448 days after casting. Fig. 16 shows the effect, c_λ, of eccentricity on the average compressive stress at failure. The tests indicated that c_λ is independent of the duration of loading. Hence, values determined with particular care in short-time tests were used to develop the relationship shown in Fig. 16.

Fig. 17 reproduces results derived from Eq. (2) for two groups of concentrically loaded test specimens ($\lambda = 0$; $c_\lambda = 1.0$), loaded at ages

of 10 days and 1 year. The selected logarithmic scale provides a good general trend, but does not reveal the details of strength changes occurring immediately after loading. Therefore, the same values were plotted in Fig. 18 on a larger time scale and as a function of duration of load. This diagram reveals clearly that the lowest strength is attained in the young concrete after a loading duration of 6 hr, while no minimum is apparent for old concrete even after a loading duration of several years.

The interrelationship between the sustained load strength, the age of concrete at application of load, the duration of loading, and the eccentricity of the applied load has been discussed. The question of deformation existing at failure is a more difficult problem. The failure does not occur suddenly, but it is a result of a gradual destruction of the internal structure of the material accompanied by a rapid increase of the deformation as shown in Fig. 13. Hence, it is difficult to give a reliable value for the deformation at ultimate strength. The test values vary within a wide range and apply to a stage in which the deformations already have reached values that are excessive in terms of practical usefulness.

Under these circumstances it is suitable for design purposes to consider a state of deformation which precedes failure. This can be done in various ways, for example, one could consider the deformation just at the onset of its rapid increase, indicating that failure is imminent. The writer feels that a suitable choice of deformation is that which exists at one-half of the loading duration to failure in sustained load

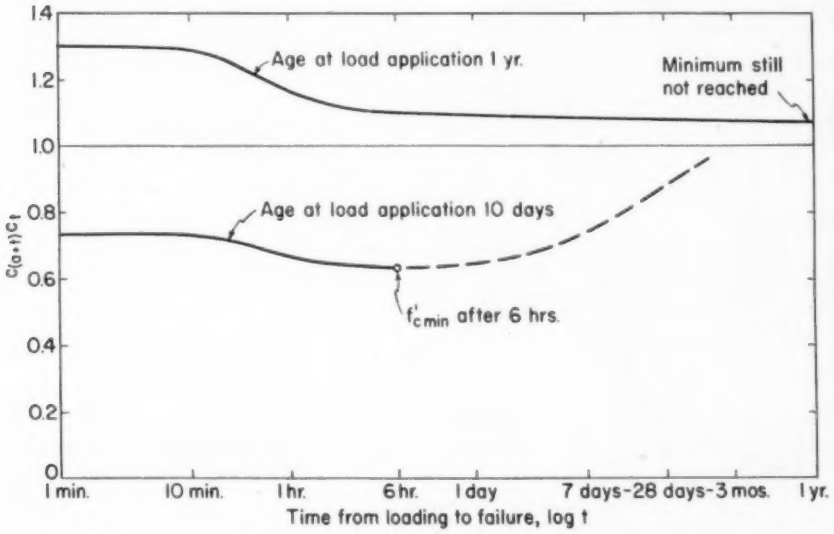

Fig. 18—Influence of time on strength under sustained concentric load

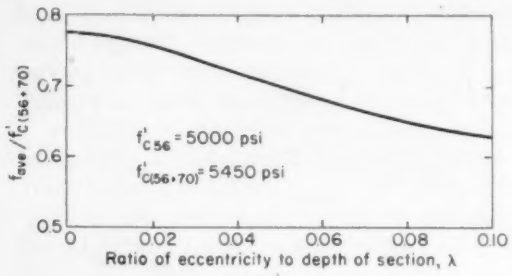

Fig. 19 — Average stress at ultimate eccentric sustained load

tests. As another possibility deformations to be used in design could be obtained by a test, in which the degree of loading is below the sustained ultimate strength, say 90 per cent.

It is not intended to discuss the advantages and disadvantages of such possibilities here. This would be worthwhile only if sufficient experimental data were available, so that the relative merits of the various methods could be evaluated. In the following, therefore, only the basic concepts of the new approach will be presented. The principal reason for selecting the deformation at one-half the time to failure is that sufficient experimental data were available from sustained load tests of 70 days duration between loading and failure.

Fig. 19 and 20 reproduce some results of the eccentric sustained load test series of concrete prisms which results are used as a basis for the present formulation of flexural theory. Fig. 19 shows the relation-

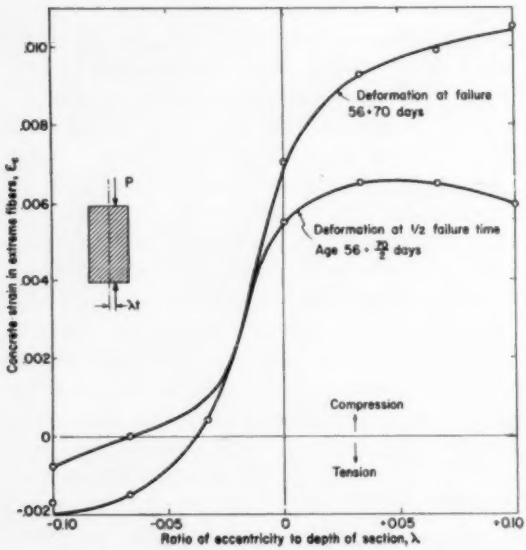

Fig. 20 — Strain in extreme fibers under sustained load

ship between load eccentricity and the average compressive stress causing failure after 70 days under sustained load. The load was applied 56 days after casting. The corresponding edge deformations are shown in Fig. 20 for load duration of 35 and 70 days, that is, for half the failure time and the complete time to failure.

In Fig. 19 and Fig. 20 the deformation developing at both edges and the corresponding average value of the stresses leading to failure after a duration of loading of 70 days can be read off for variable eccentricity. It was attempted to derive a stress-strain relationship which would correlate these test results. To this end a stress diagram limited by the two edge deformations was drawn for each value of eccentricity. The area and center of gravity of the diagram must conform to the measured values. As a first attempt diagrams were used whose upper boundaries were parabolic. The boundary lines shown in Fig. 21 fit rather well into a continuous stress-strain envelope. By trial and error better curves can be found as more test results become available. The remaining deviations will be tolerable and to a large extent attributable to unavoidable scatter of test results.

Fig. 22 presents the derived stress-strain envelope which applies for one-half the load duration in tests carried out on concrete which failed after 70 days' duration of sustained load, applied 56 days after casting.

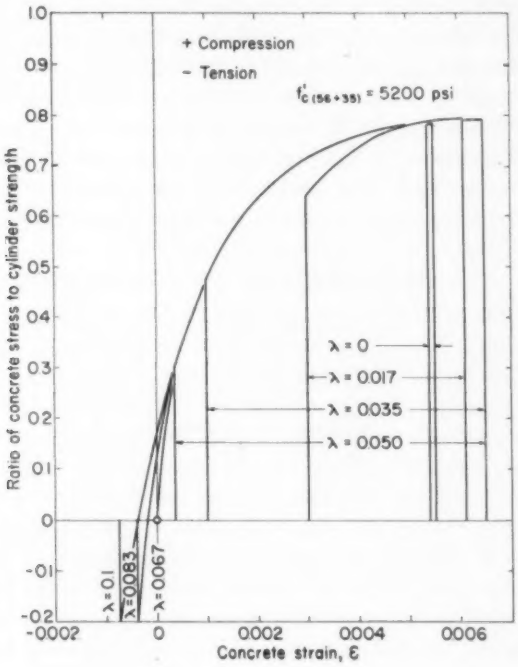

Fig. 21 — Stress-strain relationship from eccentric load tests at one-half failure time

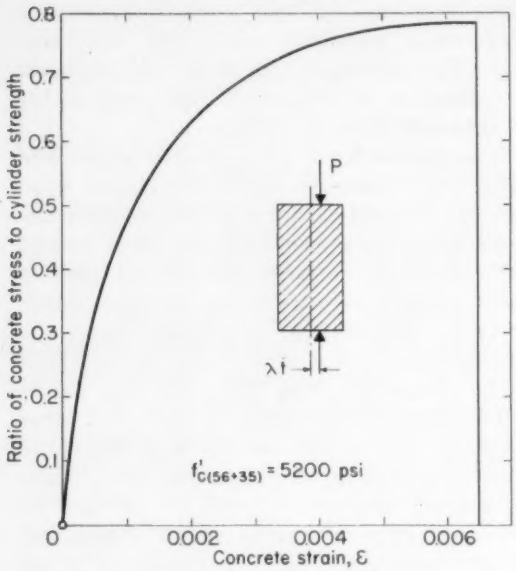

Fig. 22 — Probable stress-strain relationship from sustained load

This curve establishes another basic law of stress distribution which permits study of strength in the compression zone of structural members as a function of shape of cross section and position of the neutral axis. This can be carried out in the same manner as described in the present paper for stress-strain curves derived for constant rates of deformation.

The curve of Fig. 22 applies only to an age at loading of 56 days and a duration to failure under sustained load of 70 days. In the same manner one can, of course, by further experiments find curves which would apply to other ages, loadings, and load durations.

PRESENTATION OF THE RESULTS IN A DIAGRAM

The author suggested in 1950 [9] plotted the values required in ultimate strength design of rectangular cross sections as a function of the reduced moment of the internal concrete compressive force about the centroid of tensile reinforcement: $m_u = M_u/bd^2 f_c'$. Two coefficients are plotted as ordinates which define values at ultimate strength of the position of the neutral axis ($k_u = c/d$), the lever arm of the internal forces ($j_u = j_u d/d$), and also the strain (ε_{su}) in tensile reinforcement.

Fig. 23 shows the values of these coefficients at ultimate strength for a case of rectangular cross section and average concrete strength of 5000 psi at 400 days for constant strain rate, and at 56 days for sustained load. The diagram also takes into account effects of time. Several cases of loading at constant rate of strain were considered, the time elapsing to

failure being 10 min to 1 year. In addition, the case of sustained load applied at the age of 56 days and held constant to failure at 70 days was considered. The following comments will serve to explain the diagram:

(a) Because the values $m_u = M_u/bd^2f_c' = (M_o + P_o\ e_s)/bd^2f_c'$ were selected as abscissae,* this diagram applies to the case of pure flexure as well as to the case of combined flexure and compression. In general, low m_u values correspond to pure bending, and high m_u values to combined flexure and compression.

(b) The lowest coefficient in the diagram for the lever arm of the internal forces is $j_u = 0.50$. The resultant of the internal compressive stresses then is at the center of the cross section, that is, the case is one of a concentrically loaded column.

(c) The coefficients k_u and j_u, which determine the position of the neutral axis and the lever arm of the internal forces, are least affected by duration of loading. The coefficient k_u increases with increasing duration of loading, since the strength reduction due to time is compensated for by a lowering of the neutral axis. This leads to a reduction of the lever arm of the internal forces, expressed by a reduction of the coefficient j_u.

(d) The strain in tensile reinforcement, ε_{su}, is most strongly affected by duration of loading.

CONCLUDING REMARKS

This paper presents only an outline of a new flexural theory. The author is well aware that this work has not yet come to a decisive conclusion, and that the proposed design method does not constitute a completed solution. Thus, for example, there may be divided opinions as to the selection of the loading state from which the deformations forming the basis for sustained load design are to be determined.

Naturally it is unthinkable that practical design should involve the effect of duration of loading in detail. Not only would such a procedure be much too laborious, but it must also be admitted that in most cases it cannot be foreseen at all what service loads a structure will actually be called upon to carry during its lifetime.

Under these circumstances one is forced to start out from the least favorable conditions which can normally occur. The diagrams shown above permit us to realize clearly that these least favorable conditions occur under long-time action of a constant load.

The safety of reinforced concrete structures is generally related to their strength at 28 days. According to the reasoning followed in this paper, failure under sustained load can occur in young concrete only during the first few days, as long as ordinary portland cement is used. Furthermore, the case of young concrete being subjected to high loads is possible in most structures due to so-called construction loads.

*M_u is internal moment of concrete force about centroid of tensile reinforcement at ultimate strength; M_o is moment of external forces about centroid of section; P_o is external axial force acting in centroid of section; and e_s is distance between tensile reinforcement and section centroid.

Therefore, the requirement to design all structural units for a low age at loading, such as 28 days, is not unreasonable.

In this manner, it appears possible to replace the families of curves in Fig. 23 by a new design diagram, which presents only one curve for each of the three quantities needed in design, k_u, j_u, and ε_{su}. Tests which could be used in establishing these curves are lacking at present. However, by interpolating tests for other ages at loading, it is possible to estimate the probable shape of these curves. This is presented in

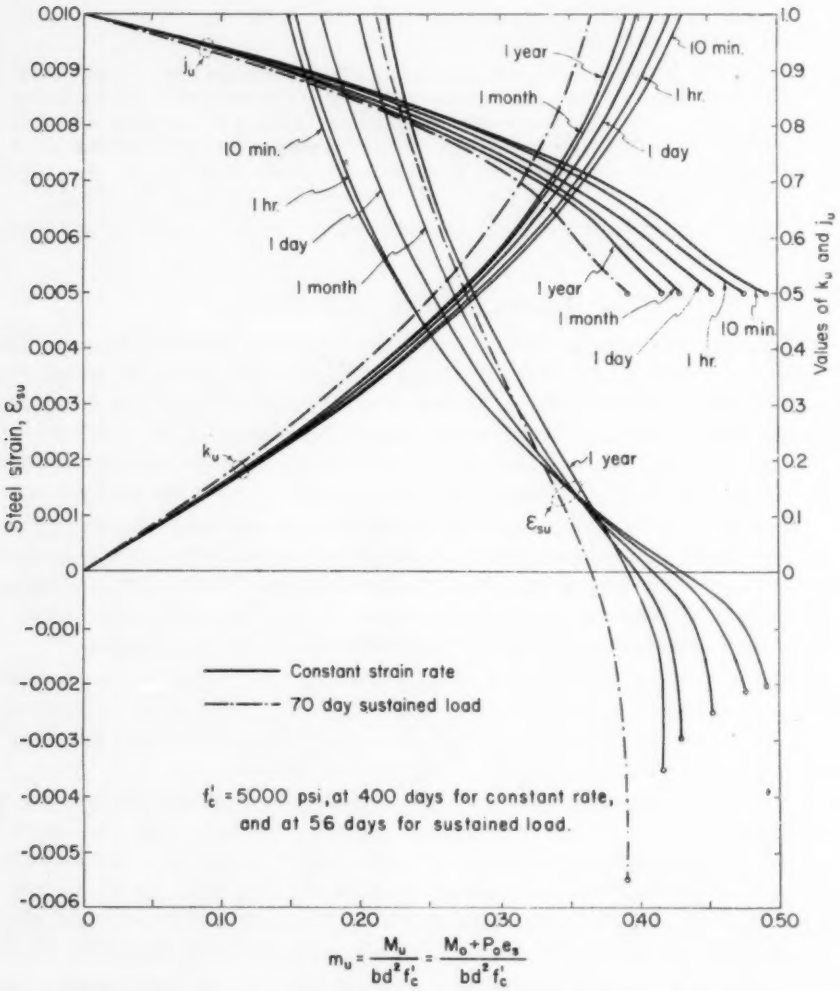

Fig. 23—Values of k, j, and ε_s at ultimate strength of rectangular cross sections

Fig. 24, although only for an average concrete strength of 4300 psi at 28 days. The final design chart should contain the three curves for each of several concrete qualities. The j_u curves will probably be so close together that they can be replaced by a single curve.

Such a diagram appears to offer the best opportunity to summarize in a simple form all the results of the theory of flexure developed here. Naturally, there are many other methods of presenting the quantities needed in everyday design, such as diagrams, tables, and simplified empirical formulas.

The advantage of a design chart of the type shown in Fig. 24 lies in its general scope of application. It embraces the entire range of

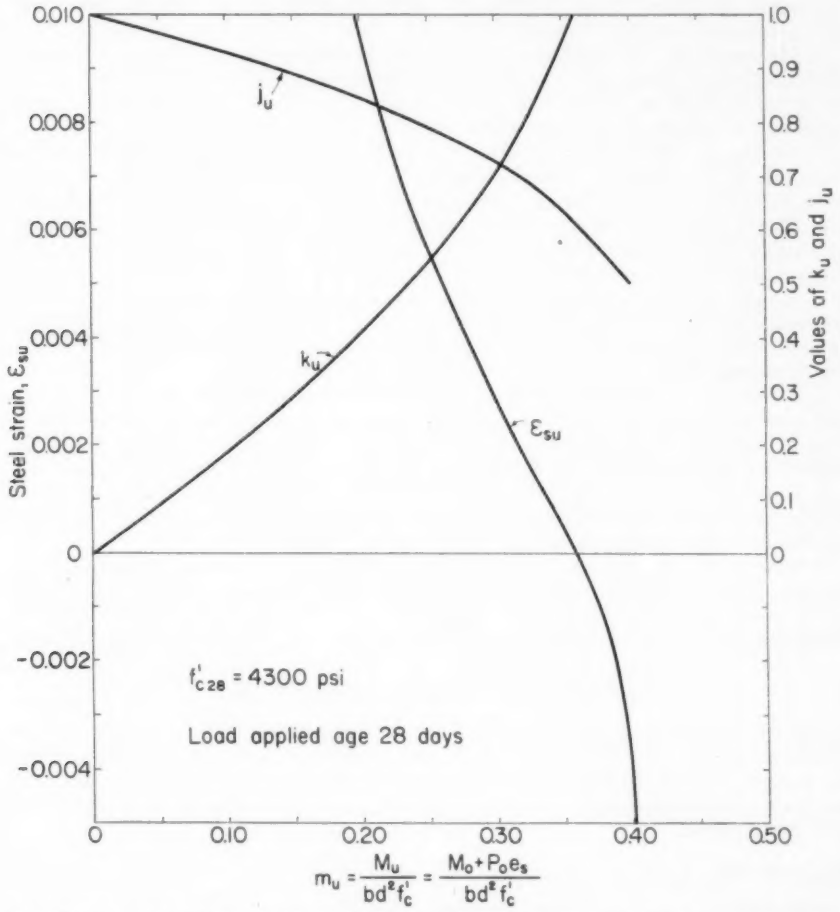

Fig. 24—Probable ultimate strength design chart for sustained load on rectangular members

stress conditions from pure bending to concentric compression. It can be used for all concrete strengths and steel qualities and applies to reinforced concrete as well as prestressed concrete. The diagram applies, however, only to rectangular cross sections, although similar diagrams can be plotted for other cross sectional shapes. The general applicability of such a diagram is illustrated in the appendix.

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APPENDIX — DESIGN EXAMPLES

The four examples below apply to a rectangular cross section with dimensions as shown in Fig. A-1. The quantities k_u, j_u, ε_{cu} needed in design are taken from the design diagram in Fig. 24. It should be emphasized again that the internal and external forces given in the examples of this appendix correspond to ultimate strength.

Example No. 1

Design load:	Pure bending; $M_o = 2620$ in.-kips
Material:	Reinforced concrete; $f_c' = 4300$ psi; cold worked deformed bars (see Fig. A-1)

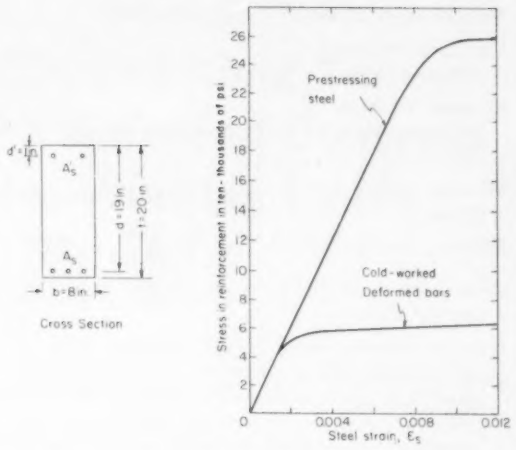

Fig. A-1—Design cross section and stress-strain curves for reinforcing steel

Relative moment: $m_u = \dfrac{M_u}{bd^2 f_c'} = \dfrac{2620}{9 \times 19^2 \times 4.3} = \dfrac{2620}{12{,}420} = 0.211$

Internal moment arm: $j_u d = 0.832 \times 19 = 15.8$ in.

Steel stress: for $\varepsilon_{su} = 0.0085$, $f_{su} = 59.8$ ksi (steel yielding)

A_s required: $\dfrac{M_u}{j_u d\, f_{su}} = \dfrac{2620}{15.8 \times 59.8} = 2.77$ sq in.

Example No. 2a

Design load: Bending and axial load acting at section centroid; $M_u = 2620$ in.-kips, $P_u = 156$ kips

Material: Same as in Example 1

Moment about tensile reinforcement: $M_u = M_u + P_u\,(d - t/2) = 2620 + 156\,(19 - 10)$
$= 4024$ in.-kips

Relative moment: $m_u = \dfrac{4024}{12{,}420} = 0.324$

Internal moment arm: $j_u d = 0.690 \times 19 = 13.1$ in.

Steel stress: for $\varepsilon_{su} = 0.00165$, $f_{su} = 46.5$ ksi

A_s required: $\dfrac{1}{f_{su}}\left(\dfrac{M_u}{j_u d} - P_u\right) = \dfrac{1}{46.5}\left(\dfrac{4024}{13.1} - 156\right) = 3.25$ sq in.

Example No. 2b

Design load: Bending and axial load acting at section centroid; $M_u = 3500$ in.-kips, $P_u = 156$ kips

Material: Same as Example 1, plus compressive reinforcement: $A_s' = 0.80$ sq in., $d' = 1$ in.

Moment about tensile reinforcement: $= 3500 + 156\,(19 - 10) = 4900$ in.-kips

Estimated position of neutral axis: $k_u = 0.80$; $c = k_u d = 0.80 \times 19 = 15.2$ in.

Tensile steel strain: $\varepsilon_{su} = 0.00162$

Compressive steel strain: $\varepsilon'_{su} = \varepsilon_{su}\frac{c-d'}{d-c} = 0.00162\frac{15.2-1}{19-15.2} = 0.0060$

Moment resisted by compressive reinforcement: $= A_s' f'_{su}\,(d-d') = 0.8 \times 58.6 \times 18 = 844$ in.-kips

Moment resisted by concrete: $M_u = 4900 - 844 = 4056$ in.-kips

Relative moment: $m_u = \frac{4056}{12{,}420} = 0.327$

New position of neutral axis: $k_u = 0.813$ (0.80 estimated k_u is all right)

Internal moment arm: $j_u d = 0.685 \times 19 = 13.0$ in.

Stress in tensile reinforcement: for $\varepsilon_{su} = 0.0015$, $f_{su} = 45.0$ ksi

A_s required:

$$\frac{1}{f_{su}}\left(\frac{M_u}{j_u d} - P_e\right) + A_s'\frac{f'_{su}}{f_{su}}$$
$$= \frac{1}{45.0}\left(\frac{4056}{13.0} - 156\right) + 0.80\,\frac{58.6}{45.0}$$
$$= 4.51 \text{ sq in.}$$

Example No. 3

Design load: Pure bending; $M_u = 3500$ in.-kips

Material: Prestressed concrete, bonded, $f_c' = 4300$ psi; prestressing steel, see Fig. A-1

Relative moment: $m_u = \frac{3500}{12{,}420} = 0.282$

Internal moment arm: $j_u d = 0.753 \times 19 = 14.3$ in.

Strain at level of reinforcement: $\varepsilon_{cu} = 0.0037$

Strain in reinforcement due to effective prestress: $\varepsilon_{se} = 0.0040$

Total strain in reinforcement: $\varepsilon_{su} = \varepsilon_{cu} + \varepsilon_{se} = 0.0077$

Steel stress: $f_{su} = 227$ ksi

A_s required: $\frac{M_u}{j_u d\, f_{su}} = \frac{3500}{14.3 \times 227} = 1.08$ sq in.

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Title No. 57-2

Instantaneous and Long-Time Deflections of Reinforced Concrete Beams Under Working Loads

By WEI-WEN YU and GEORGE WINTER

The availability of high strength steels and concretes and the acceptance of ultimate strength design have made it possible to utilize much shallower flexural members than in the past. The larger deflections of such members make it imperative for the designer to possess simple and reliable methods for pre-computing the expected deflections. This refers not only to deflections immediately upon load application, but also, and particularly, to long-time deflections under sustained loads. This paper presents two simple methods, each, for calculating instantaneous and long-time deflections under service loads. The methods are checked against the results of 175 deflection tests from nine different investigations. Agreement is found to be uniformly satisfactory for design purposes.

■ THE ADVENT OF ULTIMATE STRENGTH DESIGN and the increasing use of high strength reinforcing steels and concretes have made possible the use of shallower flexural members and have reduced the need for compression steel as compared to previously current designs. Reduction of depth and of compression steel both tend to increase short-time as well as creep deflections. It is apparent, therefore, that with the advent of these newer methods the control of deflections assumes much greater importance than heretofore.

There has been little agreement on the correct way to pre-compute short-time ("instantaneous") deflections and there has been even less information on long-time ("creep") deflections under sustained loads.

This paper presents simple methods for pre-computing the instantaneous and long-time deflections of reinforced concrete beams under working loads. Extensive comparisons of calculated and experimental results indicate the degree of agreement with 90 test results from six different investigations for instantaneous deflections, and with 85 test results from five different investigations for creep deflections.

It is shown that for both types of deflections the maximum deviations for most of the 175 tests are confined to the range of ± 20 percent with standard deviations ranging from 0.09 to 0.15 for the various methods. This is believed to be more than adequate accuracy for deflection pre-

dictions. Most of the deflection test results with which the proposed simple formulas are compared, have been obtained by other investigators. However, in regard to creep deflections, and to a lesser extent to instantaneous deflections, available evidence was chiefly confined to rectangular beams. For this reason a series of 12 tests have been carried out at Cornell University on T-beams, six under short-time loading and six under sustained loading, whose results are included in the test evidence.

The "working loads" to which the proposed deflection formulas apply are those computed on the basis of ultimate strength design, with a load factor of 1.8, corresponding to the appendix of the 1956 ACI Code.

The methods here presented are intentional simplifications of rather complex processes. They are meant to serve as practical engineering tools. More fundamental studies of the nature of reinforced concrete beam deflections, particularly under sustained loading, will be published by the first of the two authors in a separate paper.

Appendix 1 presents the derivations of the correction factor in Method B and of the "modified sustained modulus of elasticity" in Method C. Appendix 2 reports on the Cornell University tests of instantaneous and creep deflections of T-beams.

Notations

A_s = cross-sectional area of tension reinforcement
b = width of rectangular beam or flange width of T-beam
d = depth from compression face to centroid of longitudinal tensile reinforcement
E_c = secant modulus of elasticity of concrete
E'_{ct} = modified sustained modulus of elasticity of concrete at time t
f_c' = cylinder compressive strength of concrete
f_y = yield stress of the reinforcing steel
H = total depth of rectangular beam or T-beam
I_{cr} = moment of inertia of cracked transformed section
k = ratio of depth of neutral axis to effective depth of beam
L = length of beam span
M = bending moment
M_d = bending moment at design load
n = modular ratio (E_s/E_c)
n'_{ct} = sustained modular ratio (E_s/E'_{ct})
p = ratio of tensile reinforcement (A_s/bd)
δ_c = instantaneous deflection of the beam
δ_{ct} = long-time deflection of the beam

METHODS

Instantaneous deflections

Two simplified engineering methods for calculating instantaneous deflections are:

Method A—The instantaneous deflection of reinforced concrete beams under working loads can be computed by using the usual elastic deflec-

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tion equations, in which E and I are determined as follows: $E_c = 1000 f_c'$ is satisfactory for deflection calculation at design loads and for I the moment of inertia of the cracked transformed section is to be used.

This conclusion, except for the value of E_c, is identical with that obtained by Guralnick from evaluating previous T-beam tests at Cornell University.[3] For the secant modulus at design load, E_c, more accurate expressions than 1000 f_c' are available. It is found, however, that short-time deflections are quite insensitive to moderate changes in the value of the concrete modulus. To simplify matters it is, therefore, proposed that the ACI Code value, 1000 f_c', be used in deflection calculations. If this value is changed in future code editions, the revised values should then be used, and will be found to give practically identical calculated deflections.

Method B—The use of the cracked, transformed section in Method A implies that the entire concrete on the tension side does not contribute to the rigidity of the member. This is evidently only an approximation since the concrete between tension cracks does add to the rigidity. In Method B a correction factor is employed to account for this influence. The derivation of this factor is given in Appendix 1.

Consequently, the instantaneous deflection of reinforced concrete beams under design loads may be computed alternatively by multiplying the value obtained from Method A by a correction factor, $[1-(bM_1/M)]$. Here b is the width of beam at tension side in inches, M_1 is defined as:

$$M_1 = 0.1\ (f_c')^{2/3}\ H\ (H-kd)$$

where H is the total depth of the beam. The value of M_1 may be read from Fig. 1.

Long-time deflections

Two simplified methods were developed empirically from tests of 68 beams:

Method C—The total long-time deflection of reinforced concrete beams, including the instantaneous deflection and the creep and shrinkage contributions, can be computed by using in the usual elastic deflection for-

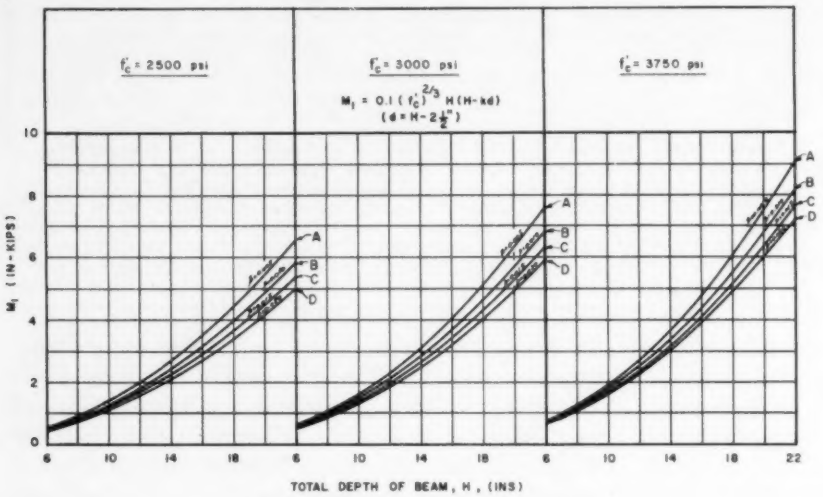

Fig. 1a—Values of M_1 for beams

A—$p = 0.005$ B—$p = 0.010$ C—$p = 0.015$ D—$p = 0.020$

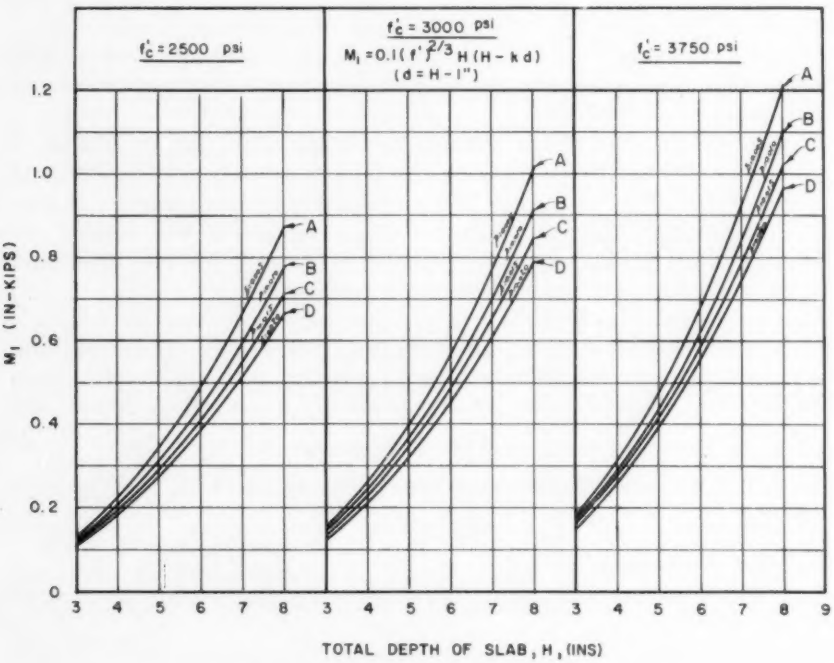

Fig. 1b—Values of M_1 for slabs

A—$p = 0.005$ B—$p = 0.010$ C—$p = 0.015$ D—$p = 0.020$

TABLE 1—MODIFIED SUSTAINED MODULUS OF ELASTICITY FOR LONG-TIME DEFLECTION CALCULATION (10^6psi) (E'_{ct})

Duration of loading	f'_c, psi	2000			2500			3000			3750			5000		
	Loaded age, days	7	14	28	7	14	28	7	14	28	7	14	28	7	14	28
	30 days	0.55	0.66	0.79	0.58	0.71	0.86	0.60	0.75	0.91	0.63	0.79	0.97	0.66	0.83	1.04
	90 days	0.41	0.51	0.63	0.43	0.54	0.67	0.45	0.56	0.70	0.46	0.58	0.73	0.47	0.61	0.77
	1 year	0.28	0.36	0.44	0.29	0.37	0.47	0.30	0.38	0.48	0.30	0.39	0.50	0.31	0.40	0.51
	3 years	0.23	0.29	0.37	0.24	0.30	0.38	0.24	0.31	0.39	0.24	0.31	0.40	0.25	0.32	0.42
	5 years or more	0.22	0.28	0.35	0.22	0.29	0.37	0.23	0.29	0.38	0.23	0.30	0.39	0.24	0.31	0.40

mulas a "modified sustained modulus of elasticity," E'_{ct}, for the specific creep conditions. This modified, sustained modulus of elasticity, E'_{ct}, has been obtained by adopting a Shank-type expression for the creep strain, and adjusting the constants in that expression to fit the results of the 68 creep deflection tests. Details of this development are given in Appendix 1. The moment of inertia of the cracked transformed section, I'_{ct}, is computed on the basis of the value of $n'_{ct} = (E_s/E'_{ct})$ computed from the same modified sustained modulus of elasticity.

The modified sustained modulus of elasticity is given in Table 1. The manner in which it depends on concrete strength, age at loading, and duration of sustained load can be visualized from Fig. 2.

Method D—In Method C an attempt has been made at a rational derivation of a deflection equation from a general, empirical creep expression. In ACI committee correspondence Robert Sailer suggested that alternatively creep deflections could possibly be determined as a simple multiple of the instantaneous deflections. Investigation showed that no single multiplier would be adequately accurate for the purpose. It also showed, however, that if such a multiplier is made to depend in a simple manner on duration of sustained loading and on conditions of reinforcing, a simple method for predicting long-time deflections is obtained.

Correspondingly, the total long-time deflection of reinforced concrete beams can be computed alternatively by multiplying the instantaneous deflection as calculated from Method A by a factor F. Values of F are given in Table 2 or Fig. 3, according to the duration of loading and the reinforcing condition.

Deflection of continuous beams

For continuous beams, the average moment of inertia of the positive and the negative moment regions is to be used in the deflection calculation.

Beams with compressive reinforcement

The effect of compression reinforcement is accounted for by taking $(n-1)A'_s$ in computing I_{cr} for the cracked transformed section. The appropriate value of n is to be used for short and long-time deflections, respectively.

COMPARISON OF TEST RESULTS WITH CALCULATED VALUES

The degree of agreement of the proposed simplified methods are indicated in the following comparisons with test results.

Instantaneous deflection

Method A—The comparison of observed deflections with those calculated from Method A is shown in Fig. 4 for 68 rectangular beams and 22 T-beams from six different investigations.

Table 3 contains the following data in connection with the instantaneous deflections for each of the 68 rectangular beams and 22 T-beams: author and reference, beam number, shape of beam, f_c', f_y, δ_{calc}, δ_{test}, and the ratio of δ_{calc} to δ_{test}.

In Table 3, the average ratio of calculated to tested deflections for 90 beams is 1.051 and the standard deviation is 0.151. If the beams which failed in shear or bond rather than flexure are excluded, then the average ratio for the rest of the beams is 1.081 and the standard deviations is 0.144.

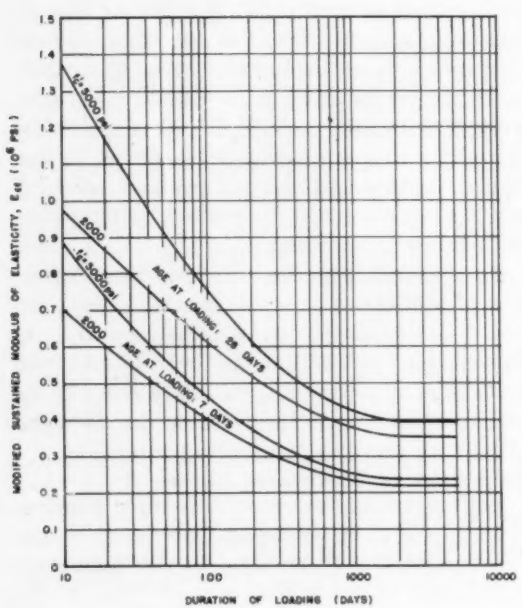

Fig. 2—Modified sustained modulus of elasticity for use with cracked section

Method B—The comparison of observed deflections with those calculated from Method B is shown on Fig. 5 and in Table 3. The agreement between calculated and test values obtained from this method is better than for Method A, but by an amount which seems to be of limited practical consequence.

The average ratio of calculated to tested deflections for 90 beams is 0.974 and the standard deviation is 0.140. But for the beams which failed in flexure, the average ratio is 1.003 and the standard deviation is 0.122.

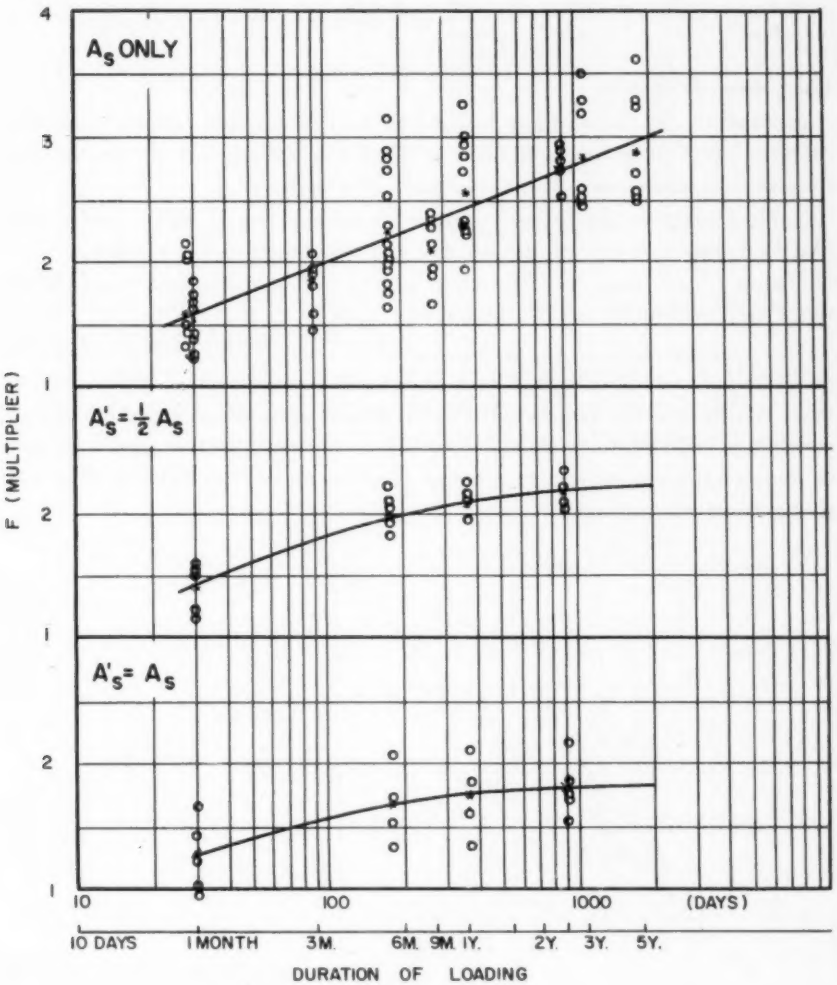

Fig. 3—Values of *F*

o—Test values
*—Average value of F at particular duration of loading

TABLE 2—VALUES OF *F* (MULTIPLIER FOR *METHOD D* FOR LONG-TIME DEFLECTION CALCULATIONS)

Duration of loading, t	F			Duration of loading, t	F		
	A_s only	$A_s' = \frac{1}{2}A_s$	$A_s' = A_s$		A_s only	$A_s' = \frac{1}{2}A_s$	$A_s' = A_s$
1 month	1.58	1.42	1.27	2 years	2.65	2.15	1.80
3 months	1.95	1.77	1.55	2½ years	2.72	2.16	1.81
6 months	2.17	1.95	1.69	3 years	2.78	2.18	1.81
9 months	2.31	2.03	1.73	4 years	2.87	2.20	1.82
1 year	2.42	2.08	1.78	5 years	2.95	2.21	1.82
1½ years	2.54	2.12	1.80				

Long-time deflection

Method C—73 rectangular beam and 12 T-beam test results from five different test programs are used to compare the calculated and experimental long-time deflections as shown in Fig. 6.

Table 4 contains the following data for each of the 73 rectangular beam and 12 T-beam observations: author and reference, beam number, shape of beam, f_c', f_y', δ_{calc}, δ_{test}, the ratio of δ_{calc} to δ_{test}, compressive reinforcement, age at loading, and duration of loading.

The average ratio of δ_{calc} to δ_{test} for 83 beams is 0.957 and the standard deviation is 0.088. For beams with only tensile reinforcement, the average ratio is 1.04, but for beams with both tensile and compressive reinforcement it is 0.88. This shows that the effect of compressive reinforcement is somewhat overemphasized in the calculation of the rigidity $E'_{ct}I'_{ct}$ in this simplified method.

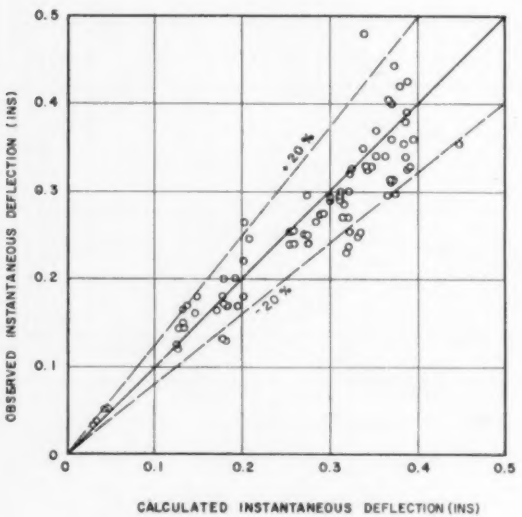

Fig. 4—Comparison of calculated and observed instantaneous deflection by Method A

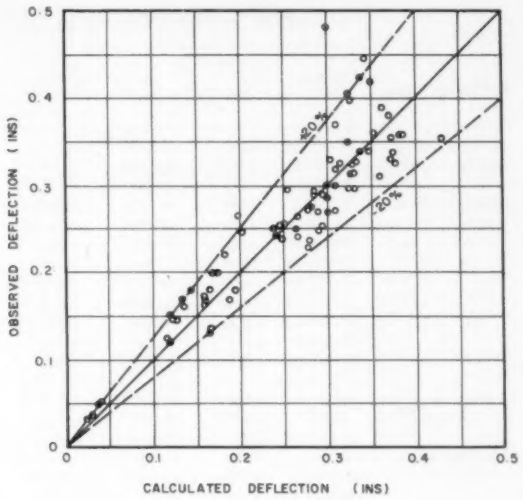

Fig. 5—Comparison of calculated and observed instantaneous deflection by Method B

Method D—The comparison of calculated and measured results is shown in Fig. 7 and listed in Table 4.

The average ratio of δ_{calc} to δ_{test} for 85 beams is 1.024 and the standard deviation is 0.102. Slightly better agreement between calculated and measured long-time deflections is obtained by using this simpler method, than by Method C.

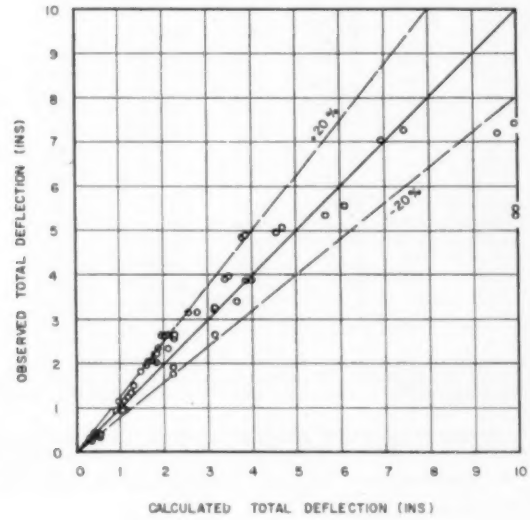

Fig. 6—Comparison of calculated and observed total long-time deflection by Method C

TABLE 3—INSTANTANEOUS DEFLECTION

Investigation	Beam No.	f_c', psi	f_y, psi	δ_{test}, in.	Method A*		Method B†	
					δ_{calc}, in.	$r = \frac{\delta_{calc}}{\delta_{test}}$	δ_{calc}, in.	$r = \frac{\delta_{calc}}{\delta_{test}}$
R. G. Mathey and D. Watstein (Reference 7)	F-IA-22	3520	42,500	0.170	0.194	1.14	0.187	1.10
	F-IA-23	3590	42,500	0.180	0.201	1.12	0.194	1.08
	F-II-7	3460	69,000	0.250	0.274	1.10	0.264	1.06
	F-II-12	4200	69,000	0.240	0.254	1.06	0.242	1.01
	F-III-8	3310	68,800	0.240	0.276	1.15	0.265	1.10
	F-III-13	4010	68,800	0.240	0.258	1.08	0.246	1.03
	F-IV-20	3380	70,500	0.255	0.259	1.02	0.248	0.97
	F-IV-21	3470	70,500	0.255	0.257	1.01	0.246	0.97
	F-VA-17	3810	98,800	0.320	0.324	1.01	0.308	0.96
	F-VA-19	3630	98,800	0.300	0.322	1.07	0.307	1.02
	F-VI-9	3730	104,300	0.350	0.338	0.97	0.323	0.92
	F-VI-18	3730	104,300	0.325	0.326	1.00	0.311	0.96
Columbia University (Reference 1)	A1	3550	66,250	0.313	0.372	1.19	0.330	1.05
	A11	3550	64,850	0.313	0.371	1.19	0.329	1.05
	A2	3550	65,630	0.296	0.375	1.27	0.331	1.12
	A12	3550	65,250	0.296	0.365	1.23	0.323	1.09
	A3	3550	66,900	0.400	0.370	0.93	0.327	0.82
	A13	3550	65,150	0.405	0.367	0.91	0.324	0.80
	B1	3550	62,430	0.355	0.448	1.26	0.428	1.21
	B11	3550	63,500	0.380	0.387	1.02	0.371	0.98
	B2	3550	62,930	0.325	0.344	1.06	0.327	1.01
	B12	3550	65,030	0.340	0.353	1.04	0.337	0.99
	B3	3550	61,430	0.340	0.363	1.07	0.346	1.02
	B13	3550	62,550	0.290	0.311	1.07	0.295	1.02
	C1	3550	61,370	0.360	0.398	1.11	0.386	1.07
	C11	3550	62,000	0.340	0.387	1.14	0.375	1.10
	C2	3550	63,040	0.325	0.389	1.20	0.376	1.16
	C12	3550	64,280	0.310	0.371	1.20	0.358	1.16
	C3	3550	60,370	0.355	0.385	1.08	0.373	1.05
	C13	3550	61,520	0.360	0.397	1.10	0.385	1.07
	D1	3510	53,000	0.230	0.318	1.38	0.277	1.20
	D11	3510	52,400	0.254	0.335	1.32	0.293	1.15
	D2	3510	53,480	0.248	0.331	1.34	0.288	1.16
	D12	3510	53,000	0.236	0.321	1.36	0.279	1.18
	D3	3510	55,100	0.200	0.302	1.51	0.260	1.30
	D13	3510	55,050	0.200	0.296	1.48	0.254	1.27
	E1	3510	56,250	0.300	0.313	1.04	0.296	0.99
	E11	3510	56,050	0.295	0.311	1.06	0.294	1.00
	E2	3510	53,860	0.294	0.301	1.03	0.285	0.97
	E12	3510	53,960	0.290	0.300	1.04	0.284	0.98
	E3	3510	55,750	0.285	0.377	1.11	0.300	1.05
	E13	3510	55,750	0.270	0.315	1.16	0.298	1.10
Portland Cement Association‡	20N	1770	44,400	0.131	0.177	1.35	0.164	1.25
	20A	1830	43,800	0.136	0.179	1.32	0.165	1.21
	40N	3900	46,900	0.168	0.170	1.01	0.159	0.95
	40A	3910	47,200	0.166	0.170	1.02	0.159	0.96
	60N	5550	58,300	0.200	0.178	0.89	0.167	0.84
	60A	5300	57,100	0.180	0.176	0.98	0.165	0.92

*Method A—Average ratio and standard deviation: (a) for all 90 beams $r_{avg} = 1.051$ and $\sigma = 0.151$; (b) for the 75 beams which failed in flexure (A-1, B-1, C-1, D-1, E-1, VA-S1, VA-S2, VA-W1, VA-W2, VB-S1, VA-S2, U-S1, U-S2, U-W1, U-W2 are excluded) $r_{avg} = 1.081$ and $\sigma = 0.144$.

†Method B—Average ratio and standard deviation: (a) for all 90 beams $r_{avg} = 0.974$ and $\sigma = 0.140$; (b) for the 75 beams which failed in flexure $r_{avg} = 1.003$ and $\sigma = 0.122$.

‡Unpublished data, Portland Cement Association, Chicago, Ill.

§Continuous beams.

TABLE 3 (Cont.)—INSTANTANEOUS DEFLECTION

Investigation	Beam No.	f_c', psi	f_y, psi	δ_{test}, in.	Method A*		Method B†	
					δ_{calc}, in.	$r = \frac{\delta_{calc}}{\delta_{test}}$	δ_{calc}, in.	$r = \frac{\delta_{calc}}{\delta_{test}}$
W. W. Yu, Cornell University T-beams (Reference 13) See Appendix 2	A-1	3680	105,500	1.340	1.189	0.89	1.120	0.84
	B-1	3880	105,500	1.240	1.176	0.95	1.100	0.89
	C-1	3530	105,500	1.190	1.181	0.99	1.110	0.93
	D-1	3680	84,200	1.270	1.288	1.01	1.240	0.98
	E-1	4260	105,500	0.510	0.577	1.13	0.530	1.04
	F-1	4260	105,500	2.200	2.040	0.93	1.890	0.86
S. A. Guralnick and G. Winter T-beams (Reference 3)	IB-1§	2440	84,610	0.150	0.133	0.89	0.120	0.80
	IIB-1§	2440	84,610	0.220	0.202	0.92	0.182	0.83
	IID-1§	5520	90,800	0.200	0.192	0.96	0.173	0.87
			84,600					
	IIIA-1§	3130	90,800	0.390	0.387	0.99	0.362	0.93
			84,610					
	IIIB-1§	3130	84,610	0.480	0.340	0.71	0.300	0.63
	IIIB-2§	3130	84,610	0.330	0.342	1.04	0.303	0.92
	IIIC-1§	5300	84,610	0.330	0.392	1.19	0.371	1.12
			83,200					
	IIID-1§	5300	90,800	0.370	0.352	0.95	0.318	0.86
			84,610					
	IA-1R	3230	90,800	0.145	0.128	0.88	0.123	0.85
	IB-1R	2440	84,610	0.120	0.126	1.05	0.119	0.99
	IC-1R	4930	83,200	0.165	0.132	0.80	0.135	0.82
	ID-1R	4930	90,800	0.125	0.122	0.98	0.116	0.93
	IIB-1m§	3240	103,100	0.172	0.178	1.04	0.159	0.93
	IID-1m§	4490	100,200	0.170	0.184	1.08	0.169	1.00
			84,200					
	IIIB-1m§	3240	103,100	0.270	0.323	1.20	0.288	1.07
	IIID-1m§	4490	100,200	0.270	0.335	1.24	0.308	1.14
			84,200					
W. McGuire and G. Winter (Reference 6)	BU-S1	3010	42,900	0.250	0.273	1.09	0.239	0.96
	-W2	3190	70,200	0.425	0.388	0.91	0.338	0.80
	BB-S1	3270	45,500	0.265	0.285	1.08	0.265	1.00
	-S2	3470	45,200	0.295	0.274	0.93	0.254	0.86
	-W1	3420	66,000	0.447	0.373	0.84	0.343	0.77
	-W2	3390	65,500	0.420	0.380	0.91	0.350	0.83
	BO-S1	2840	45,900	0.274	0.289	1.06	0.277	1.01
	-S2	2910	45,700	0.275	0.292	1.06	0.280	1.02
	BO-W1	2800	65,800	0.328	0.349	1.06	0.332	1.01
	-W2	2910	66,300	0.360	0.370	1.03	0.352	0.98
	VA-S1	3080	35,900	0.170	0.137	0.81	0.131	0.77
	-S2	2830	41,600	0.180	0.148	0.82	0.143	0.79
	-W1	2800	64,400	0.246	0.209	0.85	0.201	0.82
	-W2	3140	65,800	0.265	0.204	0.77	0.196	0.74
	VB-S1	2940	35,600	0.145	0.133	0.92	0.127	0.88
	-S2	2760	37,200	0.162	0.142	0.88	0.136	0.84
	U-S1	2900	41,000	0.034	0.029	0.85	0.024	0.71
	-S2	2750	41,200	0.036	0.033	0.92	0.028	0.78
	-W1	2660	65,500	0.051	0.043	0.84	0.036	0.71
	-W2	2780	66,100	0.052	0.045	0.87	0.038	0.73

*Method A—Average ratio and standard deviation: (a) for all 90 beams $r_{avg} = 1.051$ and $\sigma = 0.151$; (b) for the 75 beams which failed in flexure (A-1, B-1, C-1, D-1, E-1, VA-S1, VA-S2, VA-W1, VA-W2, VB-S1, VA-S2, U-S1, U-S2, U-W1, U-W2 are excluded) $r_{avg} = 1.081$ and $\sigma = 0.144$.

†Method B—Average ratio and standard deviation: (a) for all 90 beams $r_{avg} = 0.974$ and $\sigma = 0.140$; (b) for the 75 beams which failed in flexure $r_{avg} = 1.003$ and $\sigma = 0.122$.

‡Unpublished data, Portland Cement Association, Chicago, Ill.

§Continuous beams.

TABLE 4—LONG-TIME DEFLECTION

Investigation	Beam No.	f_c', psi	f_y, psi	Compressive reinforcement	Age at loading, days	Duration of loading	δ_{test}, in.	Method C		Method D	
								δ_{calc}, in.	$r = \frac{\delta_{calc}}{\delta_{test}}$	δ_{calc}, in.	$r = \frac{\delta_{calc}}{\delta_{test}}$
G. W. Washa (Reference 10)	A	4890	49,300	None	6	5 yr	7.42	9.95	1.34	6.07	0.82
	A	4890	49,300	None	6	3 yr	7.20	9.59	1.33	5.73	0.80
	B	4160	49,300	None	6	5 yr	5.50	9.95	1.81†	6.34	1.15
	C	4180	49,300	None	6	5 yr	5.33	9.95	1.86†	6.32	1.19
	D	2940	49,300	None	6	5 yr	3.28	3.14	0.96	2.95	0.90
	E	2640	49,300	None	6	5 yr	2.65	3.16	1.19	3.04	1.15
	F	2870	49,300	None	6	5 yr	3.28	3.15	0.96	2.98	0.91
	G	2750	49,300	None	6	5 yr	0.93	1.12	1.21	1.08	1.16
	H	3390	49,300	None	6	5 yr	0.93	1.11	1.19	1.02	1.10
G. W. Washa and P. G. Fluck (Reference 11)	A1 A4	3630	50,900	$A_s' = A_s$	14	2½ yr	0.93	0.91	0.98	1.10	1.18
	B1 B4	3020	47,000	$A_s' = A_s$	14	2½ yr	2.01	1.81	0.90	1.96	0.98
	C1 C4	2940	51,000	$A_s' = A_s$	14	2½ yr	3.15	2.76	0.88	3.22	1.02
	D1 D4	3220	51,000	$A_s' = A_s$	14	2½ yr	1.09	1.01	0.93	1.15	1.05
	E1 E4	2990	56,200	$A_s' = A_s$	14	2½ yr	4.88	3.84	0.79	4.09	0.84
	E1 E4	2990	56,200	$A_s' = A_s$	14	1½ yr	4.86	3.77	0.78	4.07	0.84
	A2 A5	3630	49,000	$A_s' = \frac{1}{2} A_s$	14	2½ yr	1.27	1.19	0.94	1.36	1.07
	B2 B5	3020	47,000	$A_s' = \frac{1}{2} A_s$	14	2½ yr	2.56	2.23	0.87	2.37	0.93
	C2 C5	2940	51,000	$A_s' = \frac{1}{2} A_s$	14	2½ yr	3.96	3.47	0.88	3.93	0.99
	C2 C5	2940	51,000	$A_s' = \frac{1}{2} A_s$	14	1½ yr	3.91	3.39	0.87	3.86	0.99
	D2 D5	3220	51,000	$A_s' = \frac{1}{2} A_s$	14	2½ yr	1.33	1.26	0.95	1.40	1.05
	E2 E5	2990	56,200	$A_s' = \frac{1}{2} A_s$	14	2½ yr	5.07	4.69	0.93	4.95	0.98
	E2 E5	2990	56,200	$A_s' = \frac{1}{2} A_s$	14	1½ yr	4.98	4.55	0.91	4.86	0.98
	A3 A6	3630	50,900	None	14	2½ yr	1.76	2.20	1.25	1.90	1.08
	B3 B6	3020	47,000	None	14	2½ yr	3.40	3.65	1.07	3.21	0.94
	C3 C6	2940	51,000	None	14	2½ yr	5.54	6.08	1.10	5.36	0.97
	C3 C6	2940	51,000	None	14	1½ yr	5.36	5.66	1.06	5.00	0.93
	D3 D6	3220	56,200	None	14	2½ yr	1.91	2.20	1.15	1.90	0.99
	E3 E6	2990	56,200	None	14	2½ yr	7.28	7.43	1.02	6.69	0.92
	E3 E6	2990	56,200	None	14	1½ yr	7.02	6.92	0.99	6.25	0.89
G. W. Washa and P. G. Fluck (Reference 12)	X1 X4*	3230	62,000	$A_s' = A_s$	14	2½ yr	1.14	1.06	0.93	1.16	1.02
	Y1 Y4*	3360	50,600	$A_s' = A_s$	14	2½ yr	1.81	1.48	0.82	1.76	0.97
	Z1 Z4*	3300	53,700	$A_s' = A_s$	14	2½ yr	2.32	2.09	0.90	2.32	1.00

TABLE 4 (Cont.)—LONG-TIME DEFLECTION

Investigation	Beam No.	f_c', psi	f_y, psi	Compressive reinforcement	Age at loading, days	Duration of loading	δ_{test}, in.	Method C		Method D	
								δ_{calc}, in.	$r = \frac{\delta_{calc}}{\delta_{test}}$	δ_{calc}, in.	$r = \frac{\delta_{calc}}{\delta_{test}}$
G. W. Washa and P. G. Fluck (Cont.)	X2 X5*	3230	62,000	$A_s' = \frac{1}{2}A_s$	14	2½ yr	1.27	1.14	0.90	1.39	1.10
	Y2 Y5*	3360	50,600	$A_s' = \frac{1}{2}A_s$	14	2½ yr	1.96	1.60	0.82	2.11	1.08
	Z2 Z5*	3300	53,700	$A_s' = \frac{1}{2}A_s$	14	2½ yr	2.67	2.24	0.84	2.80	1.05
	X3 X6*	3230	62,000	None	14	2½ yr	1.49	1.32	0.89	1.76	1.18
	Y3 Y6*	3360	50,600	None	14	2½ yr	2.36	1.85	0.79	2.68	1.14
	Z3 Z6*	3300	53,700	None	14	2½ yr	3.14	2.54	0.81	3.54	1.13
Portland Cement Association‡	20N	1770	44,400	None	28	9 mo	0.334	0.326	0.98	0.409	1.23
	20A	1830	43,800	None	28	9 mo	0.297	0.331	1.11	0.414	1.40
	40N	3900	46,900	None	28	9 mo	0.394	0.458	1.16	0.393	1.00
	40A	3910	47,200	None	28	9 mo	0.395	0.459	1.16	0.393	1.00
	60N	5550	58,300	None	28	9 mo	0.407	0.559	1.37	0.402	0.99
	60A	5500	57,100	None	28	9 mo	0.371	0.548	1.48	0.407	1.10
W. W. Yu, Cornell University T-beams (Reference 13) see Appendix 2	A-2	3680	105,500	None	30	5 mo	2.62	1.97	0.75	2.51	0.96
	B-2	3880	105,500	$A_s' = \frac{1}{2}A_s$	29	5 mo	2.21	1.75	0.80	2.24	1.02
	C-2	3530	105,500	$A_s' = A_s$	28	5 mo	2.03	1.62	0.80	1.96	0.97
	D-2	3680	84,200	None	31	5 mo	2.62	2.16	0.82	2.72	1.04
	E-2	4260	105,500	None	29	5 mo	1.13	0.98	0.87	1.22	1.08
	F-2	4260	105,500	None	34	5 mo	3.86	3.76	0.98	4.30	1.11
	A-2	3680	105,500	None	30	6 mo	2.65	2.02	0.76	2.58	0.97
	B-2	3880	105,500	$A_s' = \frac{1}{2}A_s$	29	6 mo	2.23	1.78	0.80	2.29	1.03
	C-2	3530	105,500	$A_s' = A_s$	28	6 mo	2.04	1.63	0.80	2.00	0.98
	D-2	3680	84,200	None	31	6 mo	2.64	2.21	0.84	2.79	1.06
	E-2	4260	105,500	None	29	6 mo	1.15	1.01	0.88	1.25	1.09
	F-2	4260	105,500	None	34	6 mo	3.95	3.86	0.99	4.43	1.14
								$r_{avg} = 0.957$§ $\sigma = 0.088$		$r_{avg} = 1.024$ $\sigma = 0.102$	

* Continuous beams.

† Not included in the calculation of r_{avg}.

‡ Unpublished data, Portland Cement Association, Chicago, Ill.

§ For tensile reinforcement only $r_{avg} = 1.04$; for both tensile and compressive reinforcement $r_{avg} = 0.88$.

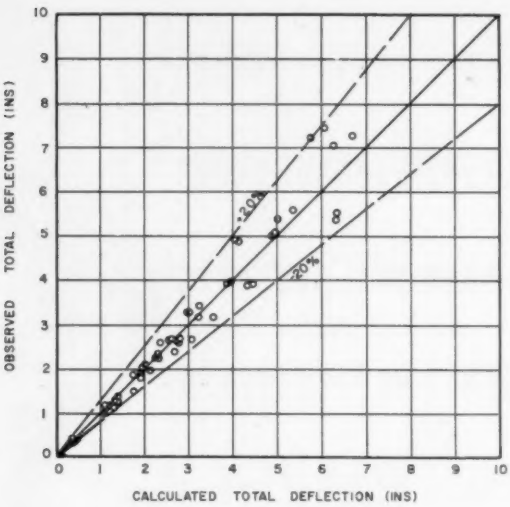

Fig. 7—Comparison of calculated and observed total long-time deflection by Method D

CONCLUSION

Two simplified methods worked out on the basis of 90 beam tests are presented for instantaneous deflections calculation. Method B provides slightly better results from the comparison of 90 calculated and experimental instantaneous deflections. The correction factor, $1-(bM_1/M_d)$, is adequate in reflecting the influence of concrete in tension on the short-time deflections.

Based on the long-time test results of 68 beams, two simplified methods are developed empirically. As compared in Table 4 and shown in Fig. 6 and 7, results obtained by using the simpler Method D are somewhat closer to the experimental values.

Fig. 4, 5, 6, and 7, as well as Table 3 and 4 show that with few exceptions all the proposed methods will predict short and long-time deflections within ± 20 percent. Considering the great variety of factors which influence deflections (strength and elasticity of concrete and steel, creep, shrinkage, degree of cracking, humidity, temperature, and others) it is believed that better accuracy can hardly be expected of a simple method. More important, it is believed that this accuracy is ample for engineering purposes. In practice it is generally irrelevant whether the deflection of a girder is ½ in. or ⅝ in.; but it is of utmost importance to know in advance whether it will be ½ in. or 1½ in.

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APPENDIX 1

Derivation of correction factor in instantaneous deflection, Method B

Assumptions of derivation—(a) Plane sections before bending remain plane after bending; (b) a straight line neutral axis can be used to represent the average of the actual, variable position of the neutral axis.

Determination of average moment of inertia—In view of tension cracks, the actual rigidity of a beam is variable along its length, being largest between cracks where the tension concrete contributes to the rigidity, and smallest

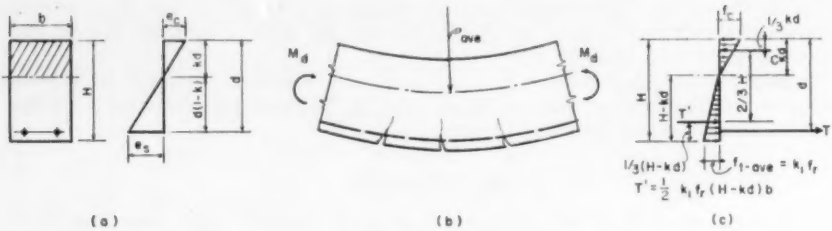


Fig. A-1

directly at a crack. Referring to Fig. A-1, let ρ_{avg} be the average radius of curvature of the beam. Then

$$\frac{1}{\rho_{avg}} = \frac{M_d}{E_c I_{avg}} \quad \text{(A1)}$$

or

$$\frac{1}{\rho_{avg}} = \frac{e_{s\text{-}avg}}{d\ (1-k)} = \frac{f_{s\text{-}avg}}{E_s\ d\ (1-k)} \quad \text{(A2)}$$

where E_c = secant modulus of elasticity of concrete
I_{avg} = average moment of inertia along the beam
$e_{s\text{-}avg}$ = average strain in reinforcement
$f_{s\text{-}avg}$ = average stress in reinforcement

To find the average stress in the reinforcement, the contributing effect of the concrete in tension has to be considered. As in Johnson's method,[4] let the average tensile stress of concrete between cracks be expressed as

$$f_{t\text{-}avg} = k_1 f_r$$

where f_r is the modulus of rupture of plain concrete.

The part of the resisting moment corresponding to the average tensile stress of concrete, $f_{t\text{-}avg}$, as shown in Fig. A-1(c), is

$$M' = T' \times 2/3\ H = 1/2\ k_1 f_r\ (H-kd)\ b \times 2/3\ H$$

$$M' = 1/3\ k_1 f_r\ b\ H\ (H-kd) \quad \text{(A3)}$$

The stress in the reinforcement corresponding to M' can be computed by

$$f'_s = \frac{M'}{A_s\ j\ d} = \frac{M'}{p\ b\ j\ d^2} \quad \text{(A4)}$$

But at the cracked section, the steel stress

$$f_s = \frac{M_d}{p\ b\ j\ d^2} \quad \text{(A5)}$$

where M_d is the moment under working loads. Then the actual average steel stress is

$$f_{s\text{-}avg} = f_s - f'_s = \frac{M_d - M'}{p\ b\ j\ d^2} \quad \text{(A6)}$$

Substituting the value of f_{s-avg} into Eq. (A2),

$$\frac{1}{\rho_{avg}} = \frac{M_d - M'}{E_s p b j d^2 (1-k)} = \frac{M_d}{E_c \left[\frac{n A_s d^2 (1-k) j}{1 - \frac{M'}{M_d}} \right]} \quad \text{(A7)}$$

Comparing Eq. (A1) and Eq. (A7), the average moment of inertia is

$$I_{avg} = \frac{n A_s d^2 (1-k)\ j}{1 - \frac{M'}{M_d}} = \frac{I_{cr}}{1 - \frac{M'}{M_d}} \quad \text{(A8)}$$

where I_{cr} is the moment of inertia of the cracked transformed section, and $1 - (M'/M_d)$ is the correction factor to account for the influence of the tension concrete.

Determination of resisting moment M' due to the tensile stress of concrete— From Eq. (A3)

$$M' = 1/3\ k_1 f_r\ b\ H\ (H-kd)$$

Many attempts have been made to express the modulus of rupture of concrete as a simple fraction of f_c'. An equation

$$f_r = k_2\ (f_c')^{2/3}$$

has been indicated as a better function by Guralnick[2] and also by a Russian text.[8] Thus Eq. (A3) can be changed as follows

$$M' = 1/3\ k_1\ k_2\ (f_c')^{2/3}\ bH\ (H-kd)$$

or

$$M' = k_3 (f_c')^{2/3}\ bH\ (H-kd) \quad \text{(A9)}$$

where $k_3 = 1/3\ k_1\ k_2$.

Sixty-eight of the short-time load beam tests have been evaluated to determine the constant k_3. The average value of k_3 was found to be 0.097 (Reference 13) but for simplicity $k_3 = 0.1$ can be used without noticeable error. Thus

$$M' = 0.1\ (f_c')^{2/3}\ bH\ (H-kd)$$

$$M' = bM_1$$

Then the correction factor of Method B is

$$\left(1 - \frac{M'}{M_d}\right) = \left(1 - \frac{bM_1}{M_d}\right) \quad \text{(A10)}$$

where

$$M_1 = 0.1\ (f_c')^{2/3}\ H(H-kd)$$

This equation has been plotted for some conditions in Fig. 1.

Derivation of modified sustained modulus of elasticity, E'_{ct} (Method C)

The value of the modified sustained modulus of elasticity, E'_{ct}, can be derived from the following equation:

$$E'_{ct} = \frac{1}{e_c + e_{shrcr}} \quad \text{(A11)}$$

where e_e = elastic or instantaneous strain for a stress of 1 psi = $(1/E_c)$ and $(E_c = 1000\, f_c')$

e_{shcr} = strain caused by creep and shrinkage for a stress of 1 psi

If we consider the shrinkage effect as a certain percentage of the creep, then the total strain due to shrinkage and creep may be computed by a modified Shank equation,[6] i.e.

$$e_{shcr} = Y\, c'\, t^{1/3} \qquad \text{(A12)}$$

where Y = multiplier according to the number of years under load[5]

t = duration of loading in days, but not to exceed 365

c' = modified coefficient of creep and shrinkage for standard portland cement, in millionths. In the present investigation this coefficient has been determined backwards from creep deflections in beam tests, by trial and error. The best fit was found to be

$$c' = 1.86c = \frac{0.93}{\sqrt[2.5]{a}} \qquad \text{(A13)}$$

Here c is the coefficient for creep deduced previously by Shank from creep tests.[5] It is defined as

$$c = \frac{0.500}{\sqrt[2.5]{a}} \qquad \text{(A14)}$$

where a is the age at loading, in days.

Substituting the value of e_{shcr} into Eq. (A11) the modified sustained modulus of elasticity is

$$E'_{ct} = \frac{1}{\frac{1}{E_c} + Yc'\, t^{1/3}} = \frac{E_c}{1 + E_c Y c' t^{1/3}} \qquad \text{(A15)}$$

Values calculated from this equation are given for various conditions in Table 1 and Fig. 2.

APPENDIX 2—REPORT ON CORNELL UNIVERSITY TESTS ON DEFLECTIONS OF T-BEAMS

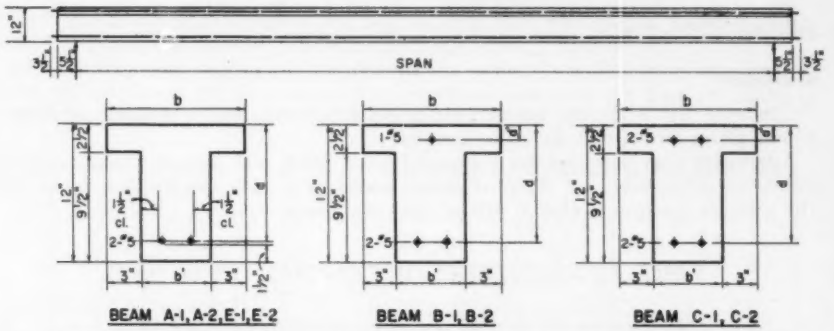
Beam specimens

The program consisted of testing 12 reinforced concrete T-beams (all dimensions are shown in Fig. A-2). Six beams (Series 1) were tested for instantaneous deflection in a uniform load testing frame. Six identical twin beams, which were tested for creep deflections (Series 2), were uniformly loaded at the service load by blocks of lead.

To reflect the effects of compressive steel, flange-web ratio (F/W), and span-depth ratio (L/D) on creep deflections, the beam specimens were designed as follows:

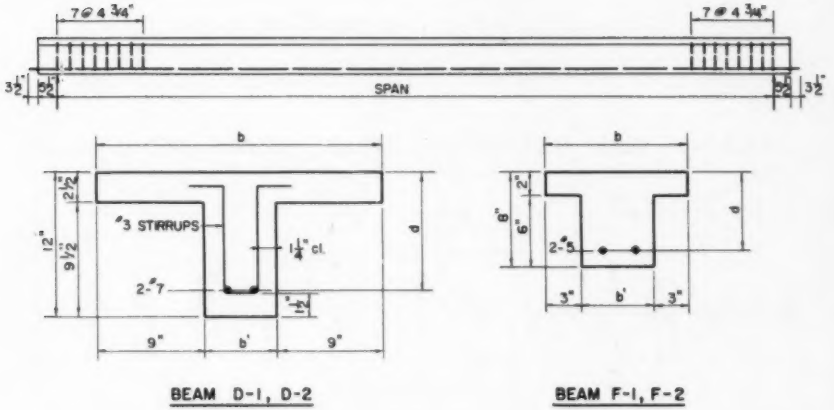
Three cases of compressive reinforcement—(a) no compressive reinforcement (Beams A-1 and A-2); (b) compressive reinforcement = 1/2 tensile reinforcement (Beams B-1 and B-2); and (c) compressive reinforcement = tensile reinforcement (Beams C-1 and C-2).

Two flange-web ratios—(a) $F/W = 2$ (Beams A-1 and A-2); and (b) $F/W = 4$ (Beams D-1 and D-2).



BEAM	SPAN (ft)	b (ins)	b' (ins)	d (ins)	d' (ins)	TOP REINFORCEMENT BAR SIZE	A'_s (IN²)	p' (%)	BOTT. REINFORCEMENT BAR SIZE	A_s (IN²)	p (%)	WEB REIN.
A-1 A-2	20	12	6	10.19	—	—	—	—	2-5	0.62	0.51	—
B-1 B-2	20	12	6	10.19	1.56	1-5	0.31	0.25	2-5	0.62	0.51	—
C-1 C-2	20	12	6	10.19	1.56	2-5	0.62	0.51	2-5	0.62	0.51	—
E-1 E-2	14	12	6	9.81	—	—	—	—	2-5	0.62	0.51	5-#3 in each end @ 4 3/4"

Fig. A-2a—Arrangement of reinforcement



BEAM	SPAN (ft)	b (ins)	b' (ins)	d (ins)	d' (ins)	TOP REINFORCEMENT BAR SIZE	A' (IN²)	p' (%)	BOTT. REINFORCEMENT BAR SIZE	A (IN²)	p (%)	WEB REIN.
D-1 D-2	20	24	6	9.68	—	—	—	—	2-#7	1.20	0.52	8-#3 in each end @ 4 3/4"
F-1 F-2	20	12	6	6.19	—	—	—	—	2-#5	0.62	0.84	—

Fig. A-2b—Arrangement of reinforcement

Three span-depth ratios—(a) $L/D = 14$ (Beams E-1 and E-2); (b) $L/D = 20$ (Beams A-1 and A-2); and (c) $L/D = 30$ (Beams F-1 and F-2).

Materials

Concrete Mix—Cement used in all beam specimens was a mixture of Huron, Alpha, Penn Dixie, and Hercules brands.

Aggregate was supplied by Rumsey-Ithaca Sand and Gravel Corp. with the following properties: (a) Sand—fineness modulus = 2.86, specific gravity = 2.67; (b) gravel—maximum size = 5/8 in., specific gravity = 2.64.

TABLE A-1—PROPERTIES OF CONCRETE AT 28 DAYS

Placement	Used in beam	Average test results			
		f_c', psi	f_r, psi	E_t, 10^6 psi	E_c, 10^6 psi
1	A-1 A-2 D-1 D-2	3679	403	3.07	2.56
2	B-1 B-2	3883	386	3.08	2.51
3	C-1 C-2	3534	397	3.05	2.50
4	E-1 E-2 F-1 F-2	4258	444	3.12	2.62

TABLE A-2—AVERAGE MECHANICAL PROPERTIES OF REINFORCEMENT

Bar size, No.	Yield stress, psi	Ultimate stress, psi	Percentage of elongation, percent
5	105,500	147,900	9.12
7	84,200	139,600	19.6

TABLE A-3—BEAM TEST RESULTS—UNDER INSTANTANEOUS LOADS

Beam	First diagonal tension crack			Failure				Computer M_u, in.-kip	Type of failure	Total service load,* lb per ft	Service live load, lb per ft	Instantaneous deflection at service live load, in.
	Jack load, kips	Maximum shear, kips	Shearing stress, psi	Jack load, kips	Maximum shear, kips	Shearing stress, psi	Positive moment, in.-kip					
A-1	0.73	8.21	154	1.01	11.01	206	660.6	826.0	D.T.	440	349	1.25
B-1	0.83	9.21	173	1.15	12.41	233	744.6	831.6	D.T.	441	350	1.15
C-1	0.73	8.21	154	1.00	10.91	204	654.6	819.8	D.T.	439	348	1.10
D-1	1.22	13.42	191	1.62	17.42	248	1045.2	915.6	D.T.	804	682	1.18
E-1	1.03	7.85	124	2.01	14.71	221	617.7	779.8	D.T.	842	752	0.49
F-1	0.72	7.83	246	1.08	11.43	351	685.8	473.3	F.T.	260	198	1.99

* Including the weight of beams.

The final proportion of water for the concrete mix was on the basis of slump tests; the slump for all mixes was in the range of 2 ½ to 3 ½ in.

Reinforcement—Longitudinal reinforcement with a nominal yield strength of 80,000 psi was used for all beams. Web reinforcement was #5 bars of intermediate grade steel with nominal yield strength of 40,000 psi.

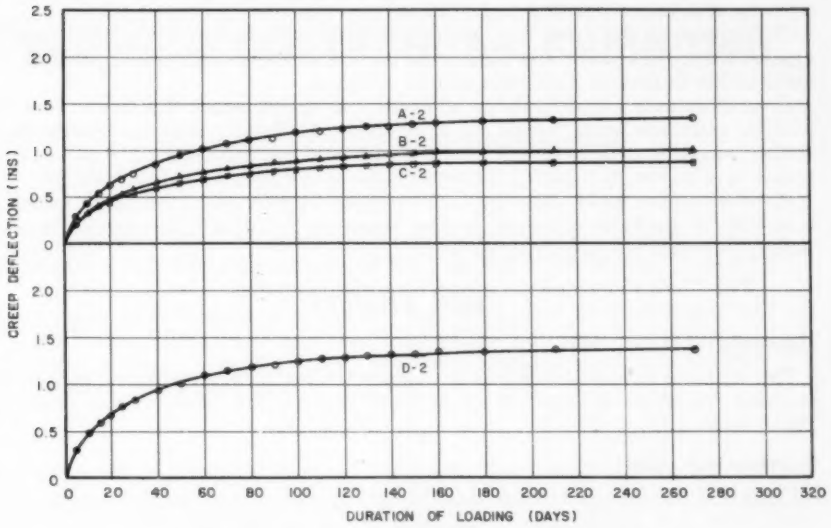


Fig. A-3a—Gross creep deflection curves

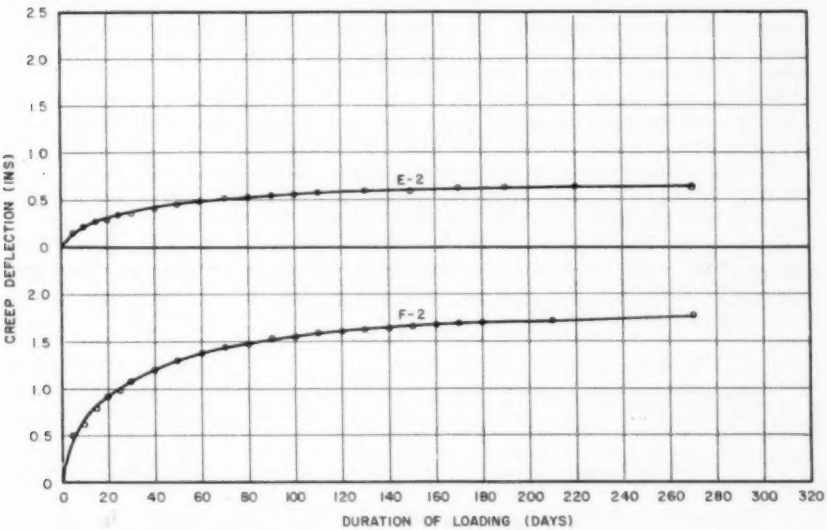


Fig. A-3b—Gross creep deflection curves

Testing equipment

Uniform-load apparatus—Beam specimens were subjected to short-time uniform loading to failure in a 30 ft long loading frame by means of 10-ton Blackhawk hydraulic rams spaced 1 ft along the beam. Sustained loads were applied by individually weighed lead pigs.

Deflection measurements—Instantaneous deflections in all beams were measured by dial gages (reading to 1/1000 in.).

Deflections in the creep test were measured with a surveyor's level reading against three permanent steel scales which were attached to angles fixed to the webs of the beams, at both ends and at midspan.

Control cylinders and modulus of rupture tests—Standard 6 x 12-in. concrete control cylinders were tested in a Baldwin 400,000 lb capacity compression testing machine. The 6 x 6 x 21-in. specimens for modulus of rupture were tested in a 12,000 lb capacity Soiltest modulus of rupture testing machine.

Reinforcing bar tests—The #7 bars used for longitudinal steel were tested in a 400,000 lb Baldwin universal testing machine. A 100,000 lb capacity Amsler testing machine was used for #5 bars.

TEST RESULTS

Concrete

The ultimate strength, modulus of rupture, initial tangent modulus, and secant modulus for stress at one-half of ultimate compression strength are shown in Table A-1.

Reinforcing steel

Table A-2 shows the average mechanical properties such as yield stress, ultimate stress, and percentage of elongation of the longitudinal steel.

Beam tests

Beams subjected to instantaneous loads—The test results of the beams subjected to instantaneous loads are reported in Table A-3. The values of M_u were computed according to the Appendix in the 1956 ACI Code, on the basis of the actual yield stress as listed in Table A-2.

Service loads were computed on the basis of the ultimate strength theory by using $f_y = 80{,}000$ psi, load factor = 1.8, and f_c' as the average compressive strength of the control cylinders.

The maximum shear and positive moment as listed include the jack load plus the dead load of the beam. Shearing stresses for Beams D-1 and E-1 were computed at sections located at the end of the stirrup group.

Creep tests—The creep tests were carried out at service load for a total testing period of 9 months under temperatures from 76 F to 82 F and relative humidities from 15 to 53 percent. Fig. A-3 shows the creep test results. For fuller details of these tests see Reference 13.

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Title No. 57-3

Design and Construction of the Civil Engineering "Arrow" at the Brussels International Exhibition

By A. PADUART and J. VAN DOOSSELAERE

Belgian civil engineering was represented at the Brussels International Exhibition by an original and audacious reinforced concrete structure. It was a reinforced concrete cantilever beam ("arrow") 270 ft long, balanced by a triangular shell roof with 95-ft sides and a thickness of 2 1/2 in. The whole construction was supported by only three columns. Architectural, structural design, and construction features are described.

■ REPRESENTING THE ARCHITECTS AND ENGINEER'S concept of the victory of civil engineering over nature, the Belgium Civil Engineering Pavilion erected for the 1958 International Exhibition in Brussels was also an effective demonstration of reinforced concrete. The reinforced concrete thin shell was designed to support an overhead passage permitting visitors to the exposition to get a birds-eye view of a relief map of Belgium open to the sun and sky. It was also intended to gracefully display an original esthetic technique.

EVOLUTION OF THE PROJECT

The form of the "arrow" resulted from a logical series of ideas based on the valuable structural possibilities of reinforced concrete. The evolution of the project is then a testimonial to this material.

The first idea was to develop an arched bridge with a span of between 120 to 150 m. Such a work would have satisfied the esthetic conditions and shown audacity, but certainly not originality.

Wishing to promote the use of reinforced concrete thin shells, the consulting engineer designed the preliminary form represented in Fig. 1. It consisted of two masts 12 m high, each carrying three horizontal arms, about 10 m long, the transverse section consisting of two shells forming a V. These two masts were 20 m apart and supported circular walkways

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at various levels. Restudying the idea, the architect utilized one of the masts to concentrate greater attention; the perspective is sketched in Fig. 2. The engineer estimated that it was possible to elongate one of the points and the draft was modified following Fig. 3. Thus, for the first time in the evolution an oblique arrow about 45 m long appeared. The other two points remaining unchanged, the mast was replaced by the springing of the arrow, completed by a forward leg providing longitudinal stability to the ensemble.

The proportions and form were again redone producing an architectural design as shown in Fig. 4; the arrow was now 63 m. This principally served as the basis for the consulting engineer's structural design; Fig. 5 represents the longitudinal section.

At this stage many modifications were again made. The length of the arrow was extended about 15 m. The central foot formed by the vertical portion of the arrow was notably lighter. At the front, the leg was doubled to form a self-stable tripod. The form of the cupola was modified so as to take advantage of surfaces of double curvature. Two oblique ties were added to accommodate the corners of the suspended room.

ARCHITECTURE

The different parts of the pavilion ensemble (Fig. 6) were to permit visitors to observe at various angles and levels the instructive and decorative mosaic map of Belgium.

The suspended room under the cupola, which constituted the counterpoise of the slanting part of the arrow, also served as the receiving hall and contained a display emphasizing the significance of civil engineering. The secondary function of the arrow was to provide a symbol of the possibilities of civil engineering.

The intention was to show that it was possible to use thin reinforced concrete shells for valuable architectural effects and to create forms submissible to logical calculation and to demonstrate the ability of the materials to be placed for their proper expression. The rough form marks in the finished concrete were intentional. The marks on the surface of the concrete and the sense of its form lumber, created with the priceless assistance of a chief carpenter of great ability, gave to the concrete great architectural significance and concurrently enhanced the function of the forms.

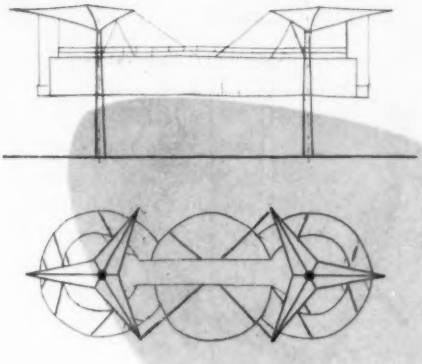


Fig. 1—Preliminary elevation (top) and plan (bottom)

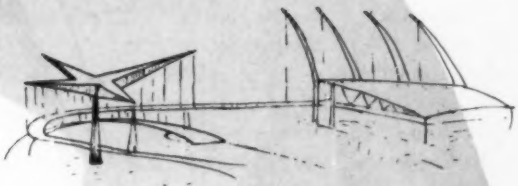


Fig. 2—Perspective of second proposed structure

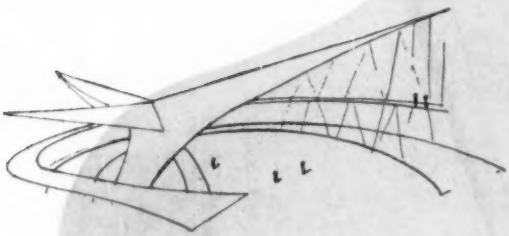


Fig. 3—Perspective of third proposal

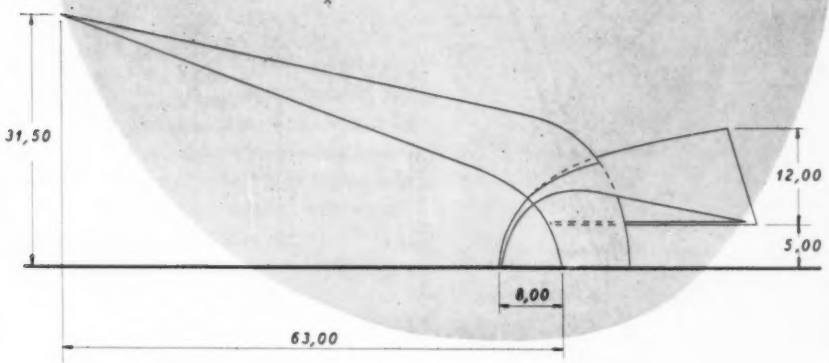


Fig. 4—Elevation of the fourth proposed project

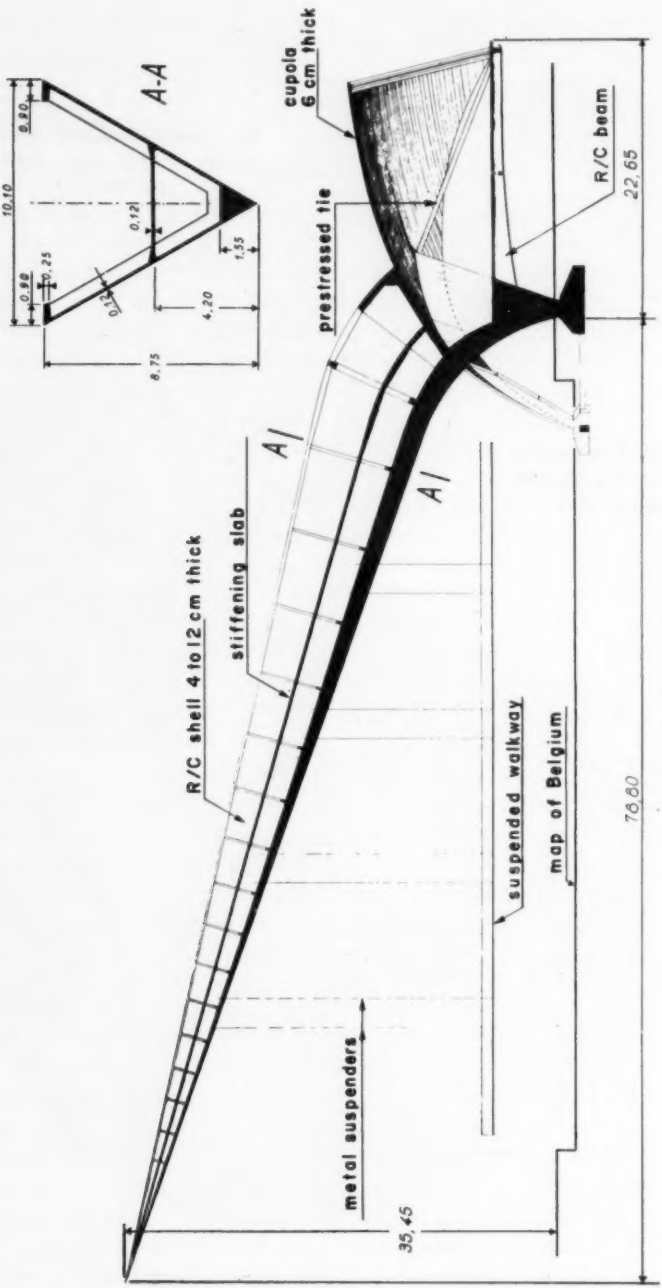


Fig. 5—Longitudinal section

CONSTRUCTION CONSIDERATIONS

A suspended foot bridge, an element designed for the circulation of the crowd, was calculated in observance of codes for the design of bridges. The live load adopted in the calculations was 400 kg per sq m with a dynamic coefficient of 1.25. On the other hand, the suspended room was in large part lined with photographic panels which would concentrate the load of the already dense crowd. For this reason a design load of 300 kg per sq m was considered sufficient for the entire surface; 500 kg per sq m being anticipated for some localized areas.

Another important consideration was the wind effect on the arrow and the cupola. As we did not have values for coefficient of friction and the lift relative to a profile with an open corner at 60 deg, tests were made at the Centre National d'Etudes et de Recherches Aéronautiques.

The components were determined for various orientations of the axis of the corner and the plane of symmetry with respect to the wind. The coefficients c_z and c_x correspond, respectively, to the vertical coefficient and the coefficient in the direction of the wind. In addition, the coefficient c_m gives the couple about the center of gravity of the triangle. These coefficients are given in Fig. 7 as functions of the angles of inclination α and β.

The actual values were obtained with the aid of these formulas:

$$Z = c_z \frac{\delta v^2}{2g} l\, a$$

$$X = c_x \frac{\delta v^2}{2g} l\, a$$

$$M = c_m \frac{\delta v^2}{2g} l\, a^2$$

where δ = specific weight of the air
v = wind velocity
g = acceleration of gravity
l = length of the angle
a = side of the equilateral triangle

The maximum carrying capacity apparently was independent of the graphics; it was obtained by applying the force of the wind normal to the plane of symmetry of the angle.

The values of coefficients determined from tests and considered in these calculations were as follows:

$c_z = 1.12$
$c_x = 1.36$
$c_m = -0.175$

Having given the dimensions of the beam, a coefficient of reduction

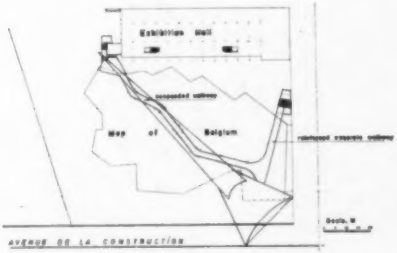


Fig. 6—Plan view of project

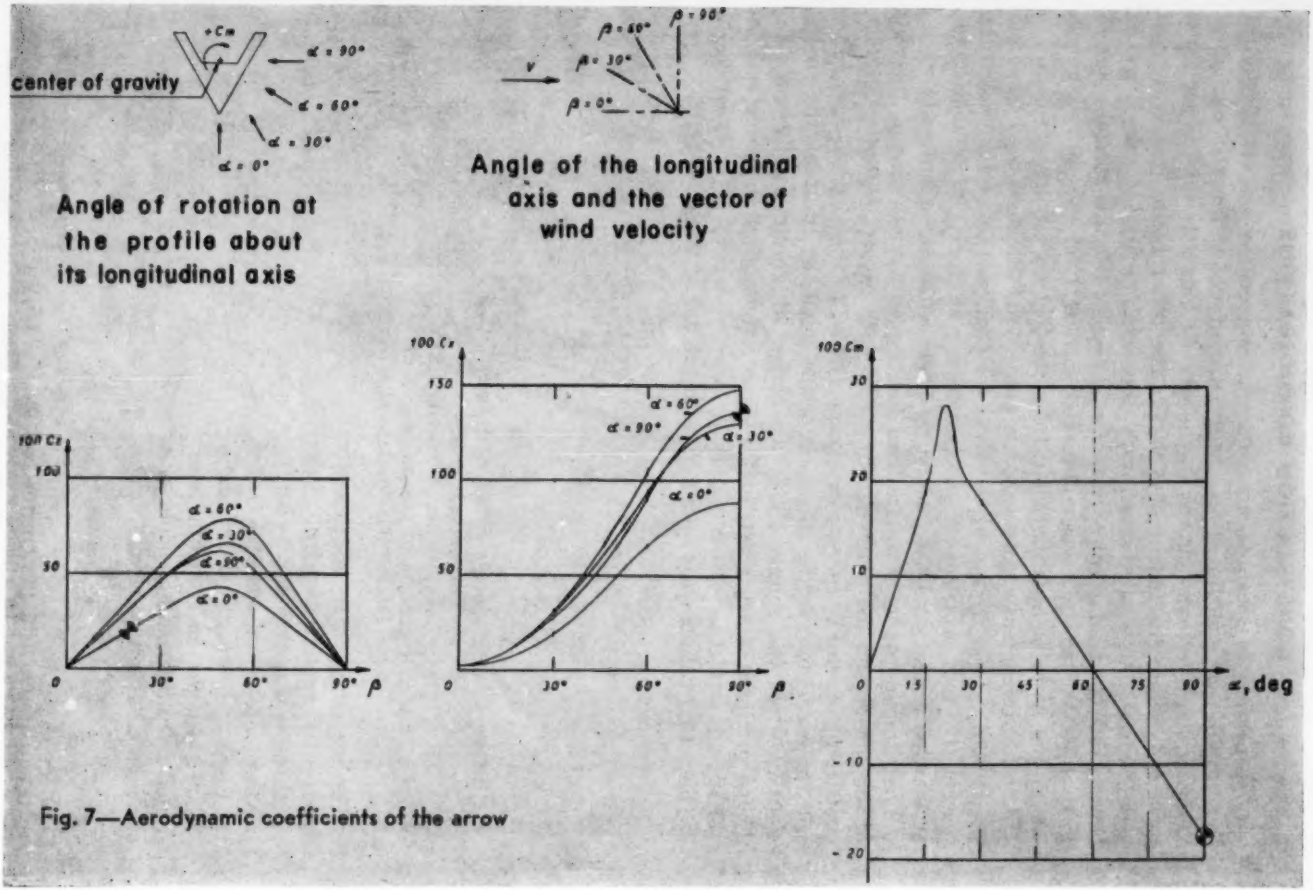


Fig. 7—Aerodynamic coefficients of the arrow

ρ was applied to the wind effects. The coefficient depends on the length of the surface acted upon and is considered to be as follows:

length, m	10	20	30	40	50	60
ρ	0.86	0.81	0.78	0.76	0.75	0.74

Next in the variable actions, the effective weight was taken into account, and was most important. The results of these calculations are reproduced later and for the fixed end moment values of the arrow, the weight of the concrete alone accounted for practically two-thirds. That is to say, in the study of forms and thickness, it was necessary to omit all volume which was not absolutely indispensable and to study the successive sections of the beam as cells of a solid of equal resistance.

ARROW AND CENTRAL FOOT

The arrow which had the primary function of supporting the foot bridge had a length of 80 m. The cross section, which appears at Section AA of Fig. 5 had the form of an inverted A, in which there were three planes or thin reinforced concrete shells. The lower line or edge of the beam made an angle of 20 deg with the horizontal and an angle of 7 deg 10 min with the plane of the top surfaces. This was considered as a solid of equal resistance, that is to say that the variation of the transverse section was such that the maximum stresses in the concrete and in the steel were practically attained simultaneously throughout the length.

The two oblique planes formed two beams resisting together all the forces passing through their line of intersection. The horizontal branch constituted a diaphragm effectively augmenting the resistance to torsion of the beam.

A graphic representation of the forces produced by the weight of the beam, by the tension of the oblique suspenders, and by wind on the beam permitted finding the most unfavorable forces arising under each plan.

The thickness of shells in the rectilinear part of the arrow varied from 4 to 12 cm. It was 20 cm thick in the curved part and thickened to a maximum of 70 cm in the central foot between the cupola and the suspended roof.

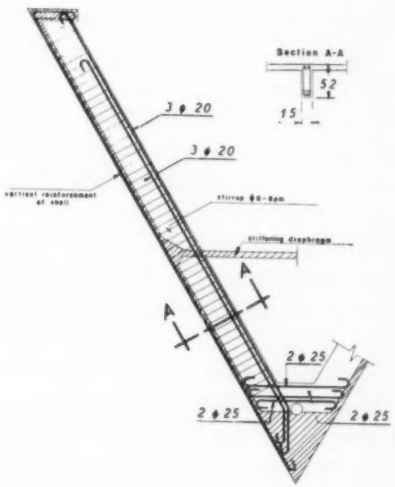


Fig. 8—Reinforcement of a transverse rib of arrow

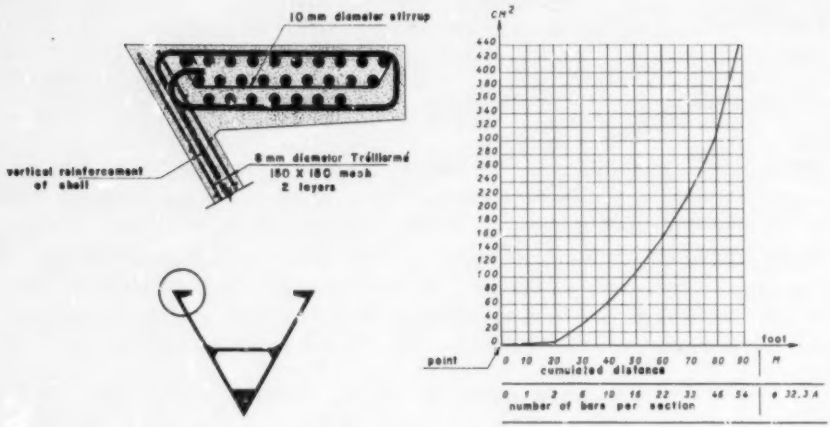


Fig. 9—Reinforcement of a top rib of arrow

The shells were stiffened by the transverse webs of which the shape and the section were increased regularly from the point toward the base, and by two longitudinal ribs along the upper edges. These two ribs, whose sections were also increasing, were designed to enclose the tensile reinforcement from the beam.

The compression member consisted of a core of triangular section of lightly reinforced concrete, of which the height was 4 cm at the point and 1.87 m in the deepest part of the arrow.

Fig. 8 shows the reinforcement of a transverse web situated 56.70 m from the point.

It is unnecessary to dwell on the concentration of steel in the longitudinal rib. In effect, in each section the surface of the concrete was reduced to that which was strictly necessary for enclosing the steel. Fig. 9 and 10 give the reinforcement of an upper rib and that required in a stiffening diaphragm, showing the disposition of the reinforcement.

For each upper rib the steel section varied from one 8-mm bar at the point to 54 deformed bars of 32.3 mm diameter of "A-52" steel. For the edges of the stiffening beam the section most heavily reinforced consisted of 30 deformed bars of the same diameter.

After placement, the bars were butt welded with an electric arc. This delicate operation was controlled by radiography and by pretested specimens. The number and length of different sections of steel to be assembled were arranged to distribute the welded joints throughout the length and to avoid eventual deformation of certain sections. The oblique walls of the beam were reinforced with 8 mm and 5 mm diameter

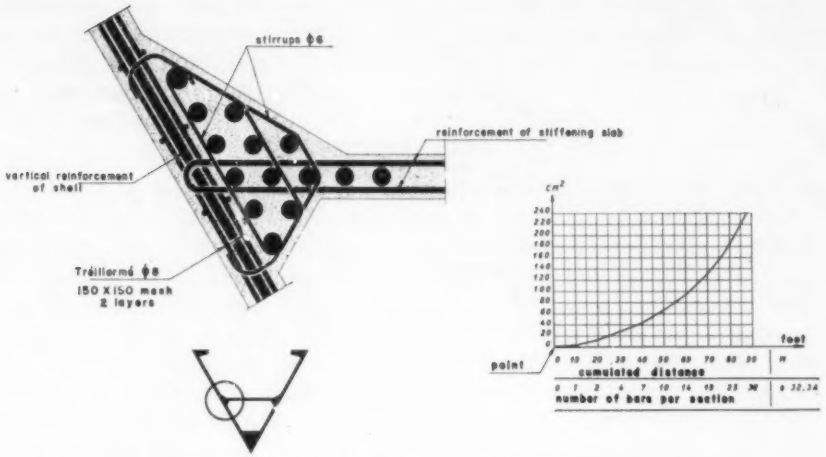


Fig. 10—Reinforcement of a gusset of the stiffening slab

"Treillarmé"; bars of 12 mm diameter were added as stirrups in sections where shear was high. Table 1 gives for each section of the arrow the most interesting information concerning the design. The abscissas were computed starting at the point of the arrow. The reduced shear T_r took advantage of the variation in height of the beam.

The moment of inertia was calculated neglecting the concrete tension and taking $m = 15$ for the section of the reinforcement. The stress σ_b'

TABLE I — DESIGN INFORMATION

Abscissa, m	10.00	20.00	30.00	40.00	50.00	60.00	70.00	80.3	85.0
M, due to the footbridge, m-tons	.....	.....	198.4	594.2	1191.8	2106.1	3098.0	4163.0	4565.0
M due to weight, m-tons	6.10	53.65	203.75	538.7	1176.9	2307.0	4121.0	7056.0	8573.0
M, total, m-tons	6.10	53.65	402.15	1132.9	2368.7	4413.1	7219.0	11,219.0	13,138.0
T, total, tons	1.88	8.69	50.0	100.3	169.0	240.0	347 0	456.7	475.0
M/r tons	0.61	2.68	13.4	28.3	47.4	73.6	103.3	141.0	156.0
T_r, tons	1.27	6.01	36.6	72.0	121.6	166.4	243.7	315.7	319.0
T_r in each wall, tons	0.73	3.47	21.1	41.5	70.2	96.1	140.9	182.3	184.4
Height of section, m	1.25	2.50	3.75	5.00	6.25	7.50	8.75	9.85	9.85
Moment of inertia, m⁴	0.0096	0.0950	0.6761	2.534	6.727	14.918	29.068	52.63	56.50
v', m	0.538	1.075	1.60	2.25	2.70	3.25	3.60	4.00	3.95
σ_b', kg per sq cm	34.2	60.7	95.0	100.8	95.0	96.3	89.6	86.1	93.4

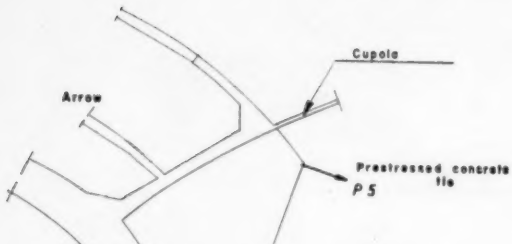


Fig. 11—Reactions at central foot

was the maximum value; it occurred at the lower surface of the arrow. The stress varied from 90 to 100 kg per sq cm throughout most of the length. It should be noted that the maximum stress in A-52 deformed steel is 2100 kg per sq cm.

In the neighborhood of the encasement, the beam was curved. This resulted in an effective tension in the vertical plane of the two semicircular upper membranes. To oppose cracking of the section resulting from this tension, the upper membranes were relieved by a tie and by a shell, which appears on the longitudinal plan in Fig. 5. The tie was computed for a maximum force of 79.5 tons and was reinforced with eight deformed bars of 32.3 mm diameter, A-52 steel.

The level of the diaphragm was necessarily arbitrary. The position chosen was a compromise between the two following circumstances: If one lowers the slab, the forces each is called upon to resist are appreciably increased and the resistance to torsion of the beam is diminished. On the other hand, if the diaphragm is placed too high its weight and its length would be increased.

Circular openings were made in the diaphragm. In addition to lightening the construction they permitted access to the bottom of the beam and thus assisted in removal of formwork from the interior of the arrow.

Rain water which fell in the arrow was evacuated by a conductor in the upper face of the base core. This drain ran through the transverse web as shown in Fig. 8. It terminated in a pipe leading into the central foot and the foundations and was bolted to the drainage system.

As indicated above, the arrow was curved to its base and prolonged by a foot of triangular cross section. Below the floor of the suspended room the foot was massive and the smallest section was a triangle 3.13 m at the base and 1.50 m high.

Deformed bar reinforcement of the arrow descended into the central feet and was terminated when no longer needed, following the diagram of bending moments. Fig. 11 indicates the various calculations which were considered in the design of the central foot.

SUSPENDED ROOM

The floor of the suspended room formed a counterpoise in the form of an equilateral triangle 29.29 m per side. It consisted of a slab of reinforced concrete 12 cm thick supported by six beams placed radially (Fig. 12). These beams, whose total height varied from 0.85 to 2.00 m, sprang from the central foot into which they were tied. In the beginning it was proposed to carry all of the load by this plan but the necessary height of beams at the intersection with the foot approached 4.50 m. The mass thus created was completely disproportionate; this solution was also abandoned for esthetic reasons.

A hinge was therefore introduced in each of the six radiating beams at the midpoint of their length. The length of brackets was thus reduced to a minimum but there was posed the problem of supporting the ends of the beams. For this purpose an edge beam (P4 in Fig. 12) was added to constitute a lower chord of a bowstring beam placed in the oblique

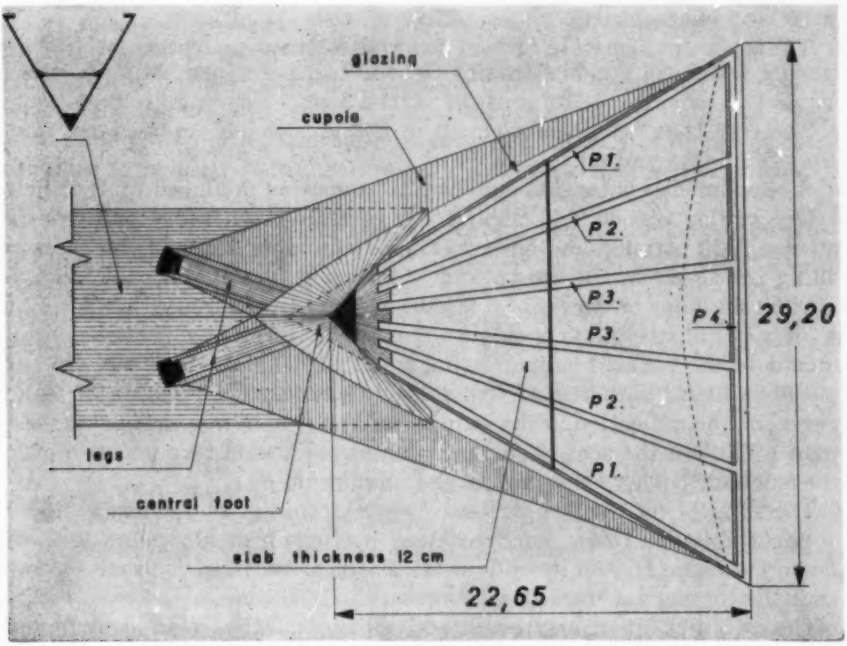


Fig. 12—View of underside of suspended room

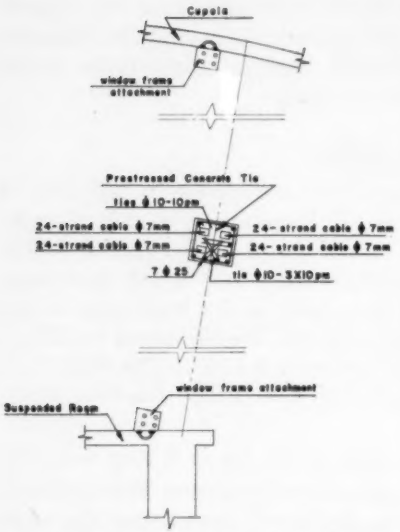


Fig. 13—Section of lateral glazing and prestressed concrete tie

plan of the window frame. The aluminum window mullions, spaced at 1.00 m, formed the suspenders of this beam and the outer surface of the arc was materialized by the edge reinforcement of the cupola (see Fig. 5).

The bowstring beam was of the Langer type, characterized by the fact that the moment of inertia of the arched element was negligible against that of the bottom chord. In such a beam the bending moments resulting from an unequal distribution of the loads were carried entirely by the lower chord; the arched rib was designed for simple compression and its unsupported length was considered to be the length separating two neighboring suspenders.

The upper rib was calculated as an arc of two segments possessing the following characteristics: span—27.90 m, rise—12 m.

The loads applied were broken down as follows: weight of P4—930 kg per m; weight of window frames—0 to 417 kg per m; radiating beams—P1 = 12.2 tons, P2 = 23.6 tons, P3 = 21.1 tons.

The results in the lower element of P4: tension—N = 48.0 tons; flexure—M = 21.9 m-tons.

A supplementary tension equal to 28.8 tons was produced by the shell.

The section was 80 x 40 cm and was reinforced with four 32.3 mm diameter A-52 bars top and bottom. The compressive stress in the arc was 40 kg per sq cm at the crown and 53 kg per sq cm at the base. Certain glazing mullions in excess of 6 m long were reinforced to resist wind by a 200 x 15-cm steel plate enclosed in an aluminum casing, the moment of inertia of the encased column being equal to 780 cm^4. With the window mullions fitting into beam P4 under an average wind pressure of 40 kg per sq m, the deflection in the largest mullion was 18 mm, which was less than 1/700th of the span. The maximum stress was 11.1 kg per sq mm in the steel and 3.8 kg per sq mm in the aluminum.

The supports of the Langer beam were located in two exterior angles of the triangular room. Each reaction was resolved along the axes of Beam P4, Beam P1 and its oblique tie, P5, of prestressed concrete joining with the top part of the central foot.

The two ties P5 along the lateral glazing of the room contributed greatly in resisting the moment of the arrow. Each tie in a square sec-

tion of 37 cm per side was prestressed with 96-wire strands of 7 mm diameter grouped in four Blaton-Magnel cables of 24 strands (Fig. 13). It also contained seven bars of 25 mm diameter designed to resist the flexure produced by the load before prestressing.

The limits of forces for tie P5 were as follows: N minimum = 251 tons, N maximum = 324 tons.

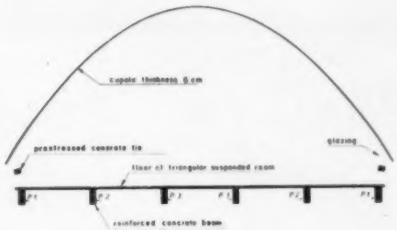


Fig. 14—Cross section of suspended room

CUPOLA AND FORWARD LEGS

The cupola which covered the suspended room was a shell of double curvature generated by the translation of a second degree parabola (Fig. 14) along the curvilinear directrix shown in Fig. 5. The calculation was made by cutting the surface into a series of arcs for each of which the apex was determined. The location of these points divided the shell into five regions: two which carried the load of the exterior angles of the room; a central triangular band exerting a force on the arrow; and finally two zones extended toward the soil in the form of the forward legs. The thickness of the cupola was 6 cm in most places; it increased to 12 cm in the neighborhood of the connection with the mullions and to 15 cm to form the arc of the Langer beam.

The two legs which, without disrupting continuity, prolonged the vault toward the foundations, were of variable section completed by a lower stem of which the form was such that the neutral axis of the global section was practically rectilinear. Fig. 15 shows a transverse section of a leg.

The maximum longitudinal compression carried by a leg was 730 tons. Including the eccentricity of the compression, the maximum stress in the concrete was 57.6 kg per sq cm.

The reinforcement included important stirrups in the wings designed to resist the bending produced by the longitudinal forces in the curved parts.

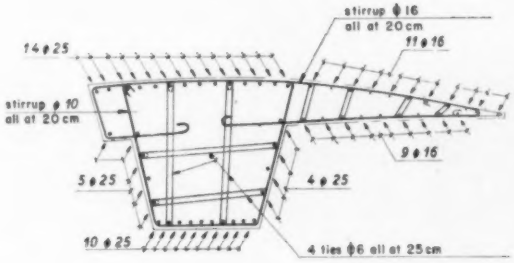
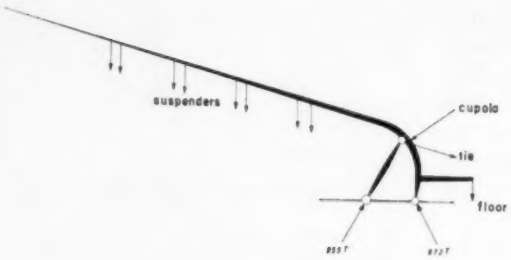


Fig. 15—Reinforcement of a leg

Fig. 16—Scheme of reactions



The section at the base of the legs formed a trapezium of which the large side was curved and in which the area was equal to 1.30 sq m.

FOUNDATIONS

The central foot and the two forward legs formed a stable tripod resting on three foundation bearings.

For calculation of the reactions, plastic joints were considered in the structure at the base of the central foot and at the upper and lower extremities of the legs.

In elevation, the assembly acted as a three-hinged arch (Fig. 16) in which one of the branches was practically rectilinear and the other was prolonged by an extremely important overhang. The design took into account the weight of the arrow, effect of the wind, tension in the suspenders of the foot bridge, load of the shell, force in the prestressed concrete ties, and weight of the floor in the suspended room.

Fig. 16 indicates the reactions corresponding to the condition when the foot bridge and the suspended room were simultaneously loaded with a crowd.

CONSTRUCTION

Earthwork and general grading began during the summer of 1956 and 28 Franki piles were placed during December of the same year. The three foundation bearings were formed, reinforced, and concreted in March, 1957.

During this time the carpenters erected a large layout floor on which all wood templates of the arrow and of the shell were manufactured. The

Fig. 17—Layout floor: preparation of template for cupola form

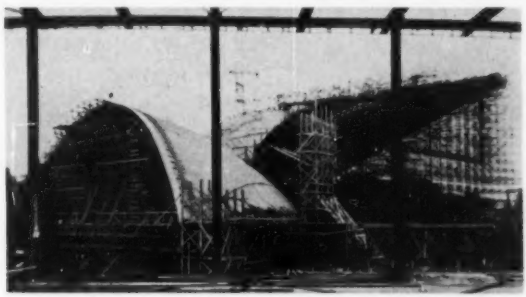


Fig. 18 — Scaffolding and formwork assembly

preparation of a template for the shell is shown in Fig. 17. The templates corresponded to the parabolic generatrices spaced at 1 m on the plan. They stood on the floor of the suspended room and served to support the formwork of the shell.

Fig. 18 is a general view of the centering for the formwork. For construction of the arrow, metallic tubular scaffolding (Fig. 19) was erected. The maximum dimensions of the scaffolding were: length—75.52 m, height—36.60 m, and width—15.50 m.

The total load of concrete supported was equal to 405 tons. There were 358 tubular posts and the total weight of the scaffolding was about 240 tons.

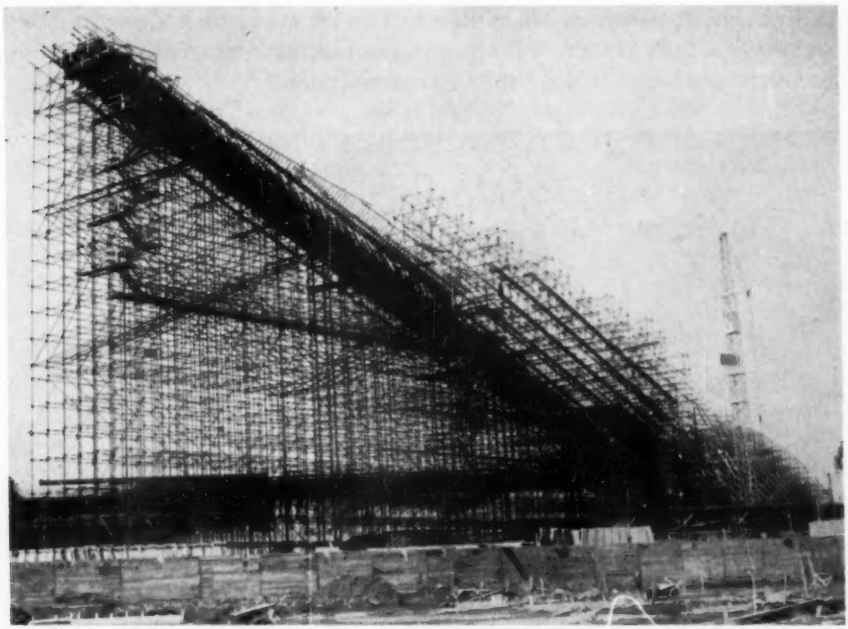


Fig. 19—Tubular steel centering for the arrow

The foundations for the tubular scaffolding consisted of unreinforced concrete footings resting on the compacted soil. The permissible load on the soil was: vertical load without wind, 0.200 kg per sq cm; vertical load with wind, 0.400 kg per sq cm.

The feet of the posts were sealed into the footings to anchor the scaffolding. In addition several guy wires were added to increase the security against rocking in case of exceptionally high winds. The floor formwork of the suspended room was also supported by tubular scaffolding.

To give maximum importance to this aspect of the construction and to accentuate the character of the reinforced concrete work, the formwork was built entirely of finished lumber. Each element of the formwork was oriented for esthetic effect in the finished concrete. The lumber was placed normal to the lower edge; also small wood laths (8 x 40 mm) were fastened horizontally on the formwork and are visible in Fig. 20. Their purpose was two-fold:

(1) Providing, after removal of formwork, grooves in the surface, making less noticeable the irregularities produced by the interruptions in concreting.

(2) Augmenting the bond between the formwork and the concrete to avoid the weight of the arrow, before decentering, coming entirely on the central tubes of the scaffolding.

These grooves, which are visible in Fig. 21, converged toward the extremity. Certain of them were stopped intermediately to make sure that the distances between diminished progressively.

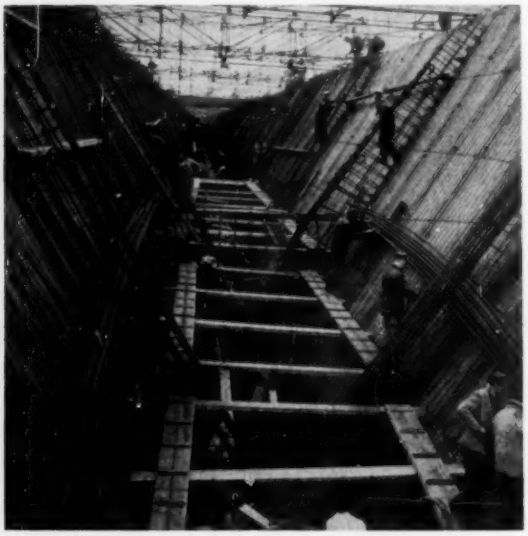


Fig. 20—Placing reinforcement in the arrow

Fig. 21—Partial view of arrow, support, and suspended walkway. Note the grooves in the arrow

Fig. 20 gives a general view of the reinforcement of the arrow. Note particularly the metallic mesh, the reinforcement of the transverse webs, and the principal deformed bars of the gussets tying into the anti-wind diaphragm. Before the reinforcement was completed it was necessary to form the interior part of the partitions and the anti-wind diaphragms.

Fig. 22 shows the reinforcement over a large part of the shell. The darker part is a double layer of reinforcement. One can distinguish the rectangular openings which were cut in the formwork to anchor the window mullions, and also the wood fastened to the formwork in the field to form the outer edge of the concrete shell.

The concrete for the arrow and the vault contained 450 kg of high-early-strength cement per cu m. Concrete was consolidated by vibrators fastened to the formwork and by internal vibrators. The compressive strength of the concrete at 28 days on 20-cm cubes was always in excess of 400 kg per sq cm. The construction was also tested with a hardness test hammer from the completion of concreting to decentering about 1 month later. The last phases of concreting were carried out with the lowest temperatures often running in the neighborhood of 0 C. Nevertheless, the work was never interrupted, thanks to the measures taken by the contractor: heating of aggregate and mixing water; protection of the fresh concrete by straw and covers; and introduction of hot air under the cupola and on the interior of the arrow.

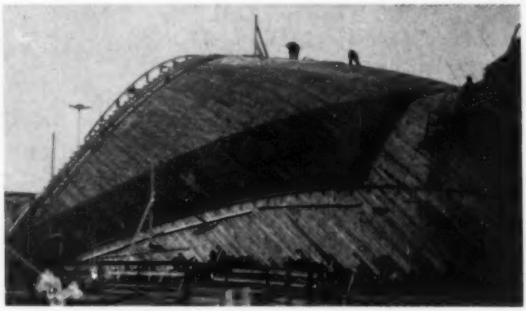


Fig. 22 — Reinforcement of the cupola

Concrete, totaling 676.9 cu m, was placed for the arrow as follows:

Foundation bearings	75.3 cu m
Arrow	256.6 cu m
Central foot	122.8 cu m
Forward legs	49.1 cu m
Shell	56.5 cu m
Suspended room	98.0 cu m
Ties	4.7 cu m
Suspended foot bridge	13.9 cu m

It is evident that because of the thin sections adopted for the major part of this construction, the proportion of reinforcement is high. The quantity of steel placed in the work equalled 119.6 tons as follows:

Steel A-37	51.0 tons
Steel A-52	57.6 tons
Treillarmé	11.0 tons

This represents about 175 kg of steel per cu m of concrete.

DECENTERING

Seven days after concreting of the point of the arrow, the forms were removed for a length of 1.50 m to check all movement of the scaffolding which would inevitably cause cracking and thus give the member low strength. This extremity (Fig. 23) was provided with a metal eyelet connected to a steel wire supporting a weight, designed to measure displacements.

The problem of deflection of a beam with a projection 80 m long was extremely important and received a great deal of attention. The elastic deformation of the arrow due to a maximum load was calculated. The moment of inertia for each transverse section was determined, neglecting the tensile concrete, and taking for the sections of steel a modular ratio of m equal to 5.25 with a modulus of elasticity for steel of 2,100,000 kg per sq cm and 400,000 kg per sq cm for the concrete. The calculation of vertical displacement effected by double integration of the diagram of elastic loads M/EI is summarized in Table 2.

TABLE 2 — VERTICAL DISPLACEMENT CALCULATIONS

Abscissa, m	I, m^4	M, m-tons	M/EI, 1/m $\times 10^4$	w, cm
0	0	0	0	50.6
10	0.00596	6.10	2.56	36.7
20	0.0634	55.65	2.115	25.3
30	0.394	402.15	2.555	16.3
40	1.538	1132.9	1.845	10.1
50	4.007	2368.7	1.479	5.44
60	8.892	4413	1.231	2.48
70	17.198	7219	1.051	0.8
80.3	31.08	11219	0.911	0

Considering the creep of concrete and the desired perspective, it was decided to give the beam a camber double that calculated. As a result, the concrete was placed 101 cm above the theoretical level.

A calculation was also done for the deflections produced only by the loads of the reinforced concrete beams and the resultant is given in

the diagram in Fig. 24. Note that the deflection of the point calculated with a modulus of elasticity of concrete equal to 400,000 kg per sq cm is 29 cm.

Fig. 23—Tip of the arrow

The operation of decentering was conducted following a conservative and detailed program and proceeded without incident during this delicate phase of the construction. The conditions were as follows:

(1) Avoid any sudden loading.

(2) Avoid all abnormal applications of loads to the arrow, the vault, and the prestressed ties.

(3) Avoid all overloading on certain parts of the scaffolding.

(4) Maintain at all times a satisfactory equilibrium between the arrow and the counterweights for the triangular room and the vault.

The principal stages of the work were as follows:

(1) Putting into each of the two beams (P5) sustaining the angles of the triangular room, 48 strands stressed at 100 kg per sq mm.

(2) Complete decentering of the vault by lowering the wood templates. Removing the formwork and shores to relieve all the forces imposed on the floor of the suspended room.

(3) Putting in each of the P5 beams 24 strands stressed at 100 kg per sq mm.

(4) Decentering the arrow by progressively lowering the tubular scaffolding.

(5) Decentering the floor of the triangular room.

(6) Putting in each of the P5 beams the remaining 24 strands stressed to 100 kg per sq mm.

Of all of these operations the decentering of the arrow was the most important.

As the presumed deflection was 29 cm and inasmuch as the screwjacks ordinarily used in tubular scaffolding have a movement not exceeding

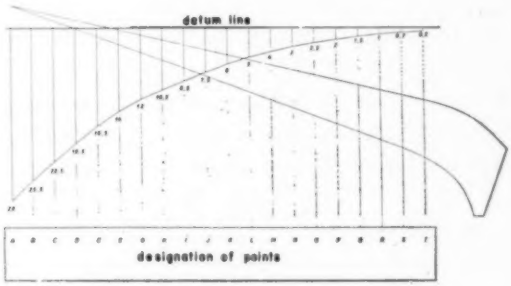


Fig. 24—Assumed deflection of arrow (ordinates are given in cm)

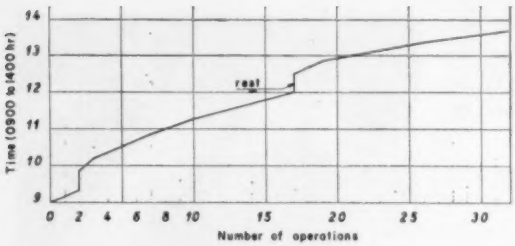


Fig. 25—Time table of decentering

19 cm it was necessary to place in each support two superimposed screwjacks. The lower screwjacks were situated at the same level about 8 m above the ground, whereas the upper jacks were placed under the supports of the joists in the formwork. The 20 sections of tubular scaffolding were designated from A to T and for each of them there was determined the presumed deflection of the arrow (Fig. 24).

The scaffolding was lowered by the lower jacks so that at each instant the descent of each head of scaffolding section was proportional to the ordinate of the curve of deformations. The number of turns to be given to each screwjack was thus variable and the work was divided in one set of operations corresponding each to three turns of the jack in section A situated under the point of the arrow. One turn of the screwjack represented a descent of 5 mm.

The arrow was decentered by a crew of 40 men divided into five groups of eight men. The progression of work is shown in Fig. 25 and it is interesting to compare this with Fig. 26 giving the deflections of the point during the course of this operation. Jacking-down took about 3 hr, at which time the beam no longer rested on the scaffolding.

The drop of the point of the arrow at the end of the decentering operation was 24 cm and it was measured each day during the following month. The variation, which was perceptibly influenced by changes in temperature, is represented in Fig. 27. Significant creep was observed following the first 15 days and amounted to a total displacement of 34 cm.

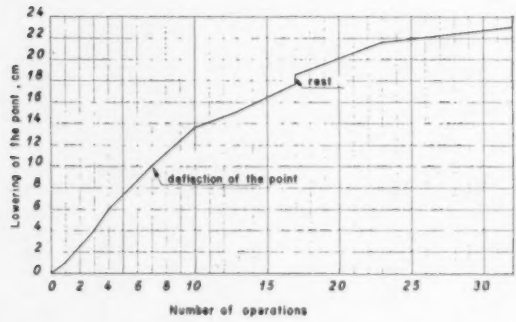


Fig. 26—Deformation during decentering

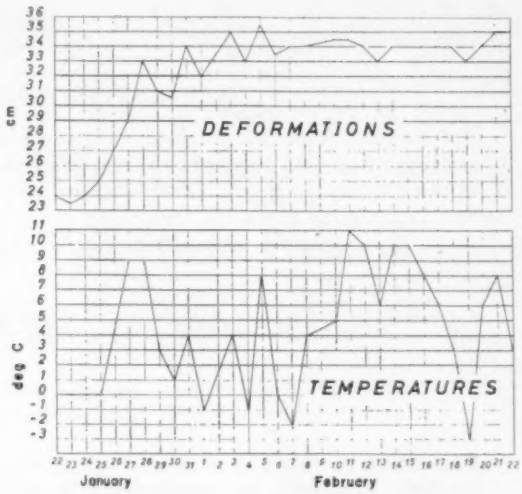


Fig. 27—Variations of the deflections after decentering

Thanks to the laths, the formwork stuck to the concrete and was detached from the branches terminating the tubular sections. The timbers were then removed progressively one by one from the concrete.

PARTICULAR CHARACTERISTICS OF THE PROJECT

1. The contrast of dimensions was exceptional for reinforced concrete, particularly when we find thicknesses of 0.04 m in contrast with the total length of 100 m.

2. The precision achieved in thicknesses of the shells reflected greater accuracy than one is in the habit of envisioning for civil engineering works.

3. The large curved surfaces gave rise to out-of-the-ordinary problems of scaffolding and forms and the placement of reinforcement.

4. The construction required the placing of materials of diverse natures which covered all the range of products utilized in civil engineering works: unreinforced concrete; tubular scaffolding; important wood carpentry; massive reinforced concrete; reinforced concrete in thin shells; prestressed concrete; metal forming; prestressing of steel; and reinforced aluminum bars.

5. This construction constituted a great experience concerning the validities of habitual methods of design, notably on the resistance and the deformation of beams of great length and height and the actions of plastic articulation based on the phenomenon of adaptation.

6. For all works of an original character, the collaboration between the architect and the engineer is indispensable from the beginning of

the studies if one wishes a solution responding simultaneously to the conditions of strength, economy, and beauty.

ACKNOWLEDGMENT

The project was the fruit of extremely close collaboration from the beginning of the studies between the coauthors with the invaluable aid of J. Moeschal, sculptor, and G. Moussiaux, at that time engineering student at the Université Libre de Brussels.

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Final report of a project sponsored by Reinforced Concrete Research Council at the University of Illinois

Title No. 57-4

Shear Strength of Restrained Concrete Beams Without Web Reinforcement

By JOHN E. BOWER and IVAN M. VIEST

Tests of two series of reinforced concrete beams without web reinforcement were made to investigate the behavior in shear of restrained beams. One series was designed as a study of the effects of variations in the ratio of maximum negative moment to the maximum positive moment (moment ratio). The other series was concerned with the effects of variations in the ratio of maximum moment to shear (moment-shear ratio $= M/Vd$).

The tests have shown that the effect of shear on the behavior of restrained beams is essentially the same as that observed for simple beams: the shear affects the load-deformation characteristics and the strength of a beam through the formation of diagonal tension cracks. The first diagonal tension crack forms at a section subjected to both moment and shear and located between the point of contraflexure and the section of maximum moment. An analysis of the test data has shown that the initial diagonal tension cracking strength is a function of the moment-shear ratio rather than of the length of the shear span.

Large variations were found in the strength beyond the initial diagonal tension cracking loads even for companion specimens. This finding supports earlier suggestions that the strength in excess of the diagonal tension cracking load is of little practical value.

Equations are presented for the initial diagonal tension cracking strength and for the shear-moment capacity.

■ SEVERAL EXPERIMENTAL STUDIES of reinforced concrete beams failing in shear were carried out in recent years, each aimed at evaluation of the effects of a few variables. The study reported in this paper was designed to clarify the behavior in shear of restrained beams without web reinforcement. In particular, it was the objective of this study to determine the effects of the negative moment over the support and of the length of the shear span on the diagonal tension cracking load and on the ultimate load.

John E. Bower, structural engineer at the Applied Research Laboratory of the United States Steel Corp., Monroeville, Pa., is engaged in studies of structural problems in steel. While a graduate research assistant at the University of Illinois, he carried out the work reported in this paper and received an MS degree in 1957. He has been a member of ACI since 1956.

Ivan M. Viest is bridge research engineer, AASHO Road Test, Ottawa, Ill. An active ACI member, he is currently chairman of ACI-ASCE Committee 333, Design and Construction of Composite Structures, and a member of ACI-ASCE Committee 326, Shear and Diagonal Tension. As staff member of the Department of Theoretical and Applied Mechanics, University of Illinois, he directed several studies of the shear strength of reinforced concrete beams and received the Institute's Wason Medal in 1955 for a paper describing one of such studies.

The investigation was carried out in two series of tests. An outline of specimens is shown in Table 1. Series I involved 20 beams in which the principal variable was the moment ratio, i.e., the ratio of the maximum negative moment to the maximum positive moment. The moment ratio was varied from zero to infinity. The tests were made on simply supported beams having two overhangs. The beams were loaded symmetrically at the ends of the overhangs and at the third-points of the span as shown in Fig. 1. The over-all length of the beams varied from 11.5 to 19 ft.

Series II involved 14 beams in which the principal variable was the moment-shear ratio, i.e., the ratio of the maximum moment to the external shear divided by the effective depth. The moment-shear ratio was varied systematically from 1.5 to 7.5. The series consisted of simply supported beams with or without one overhang (Fig. 1). The beams with overhang were loaded with one load inside and one load outside the span; the beams without the overhang were subjected to two equal loads placed at the third-points of the span. The over-all length of beams varied from 7 to 19 ft.

The variations in the moment ratio and in the moment-shear ratio were obtained through changes in the length of the shear span and in the relative magnitude of the loads placed on the overhangs and in the span.

Definitions and Notations

Moment ratio = ratio of the maximum negative to the maximum positive moment in a shear span (M^-/M^+)

Moment-shear ratio = ratio of the maximum moment to the external shear in an effective shear span divided by the effective depth of the tension reinforcement (M/Vd)

Critical section = vertical section through the edge of a load or support bearing block inside a shear span

Diagonal tension crack = an inclined crack extending from the longitudinal tension reinforcement toward the critical section and making an angle of approximately 45 deg with the longitudinal reinforcement

Shear-moment capacity = moment at shear-compression failure

a = length of the effective shear span equal to the distance from the section of zero moment to the section of maximum moment (Fig. 1); also equal to the ratio of the maximum moment to the shear in the effective shear span

a' = length of the shear span (Fig. 1); for any particular load, the external shear is constant throughout the length of the shear span

a_c = distance from the section of zero moment to the intersection of a diagonal tension crack with the longitudinal tension reinforcement

b = width of the beam

d = effective depth of the tension reinforcement

E_s = modulus of elasticity of steel

f_c' = compressive strength of 6 x 12-in. concrete cylinders

f_s = stress in the tension reinforcement at failure

k_1k_3 = parameter defining the magnitude of the compressive force resisted by concrete at the critical section [Eq. (7)]

K = a measure of the concentrated angle change [Eq. (8)]

l = span length

$(M/V)_c$ = ratio of moment to shear at the section of diagonal tension cracking

M = bending moment

M_f = moment capacity of the beam at flexural failure

M_{max} = the numerically larger of the two moments at opposite ends of a shear span

M_{min} = the numerically smaller of the two moments at opposite ends of a shear span

M_s = moment capacity of the beam at shear-compression failure

P = total load

P_1,P_2 = individual concentrated loads

P_c = total load at diagonal tension cracking

P_y = total load at yielding of the tension reinforcement

P_u = ultimate load; maximum total load reached during a test

p = ratio of tension reinforcement

v_c = V_c/bd; nominal shearing stress at diagonal tension cracking

V = shear force

V_c = external shear at diagonal tension cracking in the shear span in which the diagonal tension crack is located

V_u = external shear at ultimate load in the shear span in which failure occurred

x = distance along the beam from the section of zero moment

ε_u = ultimate strain in concrete [Eq. (8)]

SPECIMENS AND TESTS

Specimens

An outline of the test beams is given in Table 1 and Fig. 1 shows the nominal beam dimensions. The actual dimensions and properties, different from the nominal values, are given in Tables 3 and 4.

All beams had a rectangular cross section 6 in. wide and 14 in. deep. The nominal concrete strength was 3500 psi. The beams were reinforced in both tension and compression with two #7 deformed bars of intermediate grade steel extending the full length of the beam. The nominal percentage of tension reinforcement was 1.6 percent, and the nominal effective depth of both the top and bottom reinforcement was 12 in. The side cover of the longitudinal reinforcement was 1.5 in.

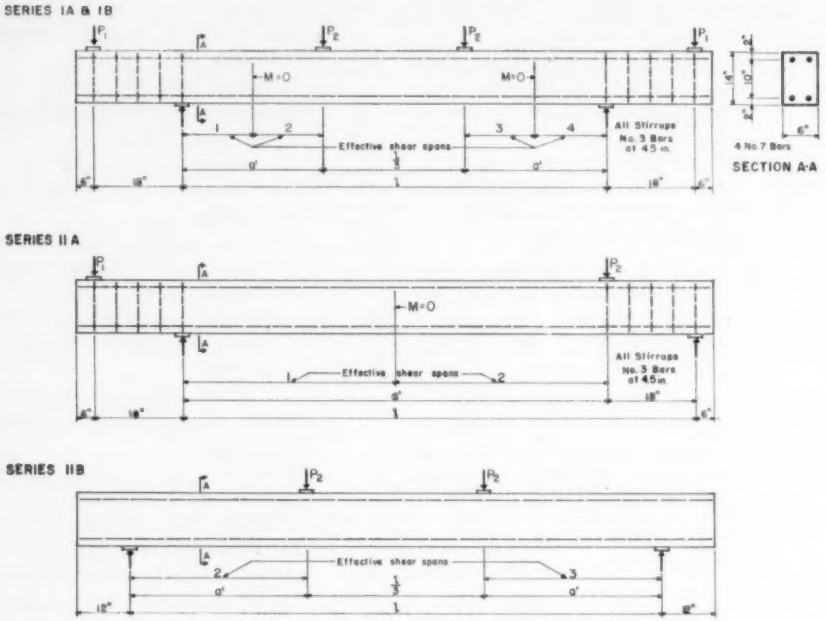

Fig. 1—Details of beams

Web reinforcement was provided where it was desired to prevent shear failure. The locations of the web reinforcement are shown in Fig. 1. Closed vertical stirrups of #3 deformed bars of intermediate grade were placed at 4.5-in. spacing. Beams of Series IIB had no web reinforcement; sufficient length of beam was provided beyond the supports to prevent bond failures.

The length of the shear span, a', varied from specimen to specimen which resulted also in variations of the span length (Fig. 1). In Series I the length of the effective shear spans, a, containing the maximum moment was constant. For Series IA, a was equal to 30 in., while for Series IB it was equal to 24 in. In Series II the length of the effective shear span was variable; for Series IIA $a = \frac{1}{2}a'$, and for Series IIB $a = a'$. The lengths of the shear span, a', are given in Table 1.

Materials

All beams were cast from concrete made with Type I portland cement, Wabash River gravel, and Wabash River torpedo sand. The cement was purchased in two lots and the aggregates in five lots. The average gradations of the aggregates are given in Table 2. The maximum size of the gravel was 1 in. and the fineness modulus of the sand was 3.23. The absorption of both aggregates was approximately 1 percent by weight.

The average proportions of cement to sand to gravel were 1:3.84:5.76 by weight and the cement-water ratio was 1.11. The average slump was 3 in. Mixing of the concrete was done in a nontilting, horizontal 6.5-cu ft drum mixer for approximately 5 min. The compressive strength, the modulus of rupture, and the initial modulus of elasticity of concrete are listed for each

TABLE 1 — OUTLINE OF BEAMS

Beams	Moment ratio M^-/M^+	Nominal moment-shear ratio M/Vd	Shear span* a', in.	$\frac{P_1}{P_2}$
		Series I		
IA -1a, b†	0.	2.5	30	∞
2a, b	0.333	2.5	40	1.80
3a, b	0.667	2.5	50	0.90
4a, b	1.000	2.5	60	0.60
5a, b	1.500	2.5	50	0.60
8a, b	2.250	2.5	43.3	0.60
6a, b	3.000	2.5	40	0.60
7a, b	∞	2.5	30	0.60
IB -1	0.333	2.0	32	2.25
2	0.667	2.0	40	1.13
3	1.500	2.0	40	0.75
4	3.000	2.0	32	0.75
		Series II		
IIA -1a, b	1.0	1.5	36	2.00
2	1.0	2.0	48	1.75
3	1.0	2.5	60	1.60
4a, b	1.0	3.0	72	1.50
8	1.0	3.5	84	1.43
9	1.0	4.0	96	1.38
5	1.0	4.5	108	1.33
6	1.0	6.0	144	1.25
7	1.0	7.5	180	1.20
IIB 1	0	3.5	42	∞
2	0	4.0	48	∞
3	0	4.5	54	∞

*See Fig. 1.
†Letters a and b denote companion specimens.

beam in Table 3. The compressive strength and the modulus of rupture are the averages for all control specimens made with the particular beam. The modulus of elasticity is the average for one control cylinder from each batch of concrete.

The reinforcing bars were of intermediate grade steel having the following average properties: yield point of 46 ksi, ultimate strength of 81 ksi, 22 percent elongation over an 8-in. gage length, and modulus of elasticity of 29.5×10^6 psi. The mechanical properties, other than the modulus of elasticity, were determined from a standard tension test of one sample from each reinforcing bar. The modulus of elasticity and the bar dimensions, listed in Table 4, were determined on samples from five bars of each size. The dimensions met the requirements of ASTM A 305-56T. The average yield points of bars for individual specimens are shown in Table 3; for most specimens, the difference between the yield points of individual bars was less than 1 percent.

TABLE 2 — GRADING OF SAND AND GRAVEL

Sieve size	Percent retained	
	Sand*	Gravel*
1½-in.	0	0
¾-in.	0	32.1
⅜-in.	0	78.9
No. 4	1.9	99.0
8	18.1	99.5
16	41.0	99.5
30	69.3	100.0
50	93.8	100.0
100	98.5	100.0
Fineness modulus	3.23	7.07

*Average for five lots.

Fabrication of specimens

All beams were cast in steel forms. For beams with web reinforcement in the end spans, the reinforcement was tied into a cage. The bottom bars were supported from the base of the forms on steel chairs. The top bars were supported temporarily by wire hangers; the hangers were removed after the concrete had been placed.

All beams required two or three batches of concrete placed in horizontal layers and vibrated with an internal rod vibrator. Spreading of the forms was prevented by external clamps. Lifting hooks were placed in the overhangs.

Four 6 x 12-in. cylinders and one 6 x 6 x 20-in. control beam were made from each batch. A few hours after casting, the test beam and its control specimens were removed from the forms and covered with moist burlap for 6 days. All specimens were then stored in the laboratory until the time of testing. The age at testing is shown for each beam in Table 3.

Testing equipment and procedures

The load was applied to the beams by a loading assembly connected to a specially built loading frame. The loading assembly consisted of a 50-ton hydraulic jack, a 125,000-lb elastic ring dynamometer, and of one or three load distributing beams (Fig. 2). The load was measured to the nearest 100 lb with a 0.0001-in. dial indicator. The strength of Specimen IC-4a exceeded the

TABLE 3 — PROPERTIES OF BEAMS

Beam	Age at testing, days	Yield point of reinforcement f_y, ksi	Effective depth of reinforcement, in.		Concrete strength		
			Bottom	Top	Cylinder strength f_c', psi	Modulus of rupture f_r, psi	Modulus of elasticity E_c, 10^6 psi
IA-1a	29	47.0	12.06	12.00*	3300	406	3.32
b	30	46.8	12.38	12.00*	4350	400	4.22
2a	30	44.6	12.25	11.50	3590	580	3.47
b	28	46.3	12.25	11.50	3540	400	3.38
3a	30	47.3	12.25	11.50	4020	450	3.88
b	28	46.9	12.00	12.00	3550	397	3.50
4a	30	46.5	12.00	12.00	3300	478	3.52
b	28	46.3	12.25	11.50	3170	394	3.04
5a	30	45.9	12.12	11.80	3460	430	3.50
b	30	46.8	12.12	11.88	3120	455	3.02
8a	30	44.5	12.25	11.50	3500	503	3.23
b	30	46.3	12.00	11.50	3120	489	3.27
6a	25	44.6	12.00	12.00	3120	390	3.33
b	30	46.8	12.00	11.75	3520	395	3.25
7a	27	47.4	12.00	12.00	3070	392	3.00
b	30	45.9	12.00	12.00	3280	479	3.52
IB-1	30	47.3	12.00	12.00	3220	454	3.06
2	30	47.8	12.00	11.75	3080	338	2.98
3	33	46.3	12.00	12.25	3310	475	3.20
4	32	44.3	12.00	11.75	3490	538	3.35
IIA-1a	31	46.3	12.12	12.75	3300	562	2.89
b	28	47.0	12.50	11.38	3700	429	3.89
2	29	46.2	12.28	11.78	3130	562	3.22
3	30	46.4	12.28	11.50	3150	491	3.28
4a	30	46.2	12.50	12.00	2900	417	3.02
b	25	46.9	12.15	11.50	3320	398	3.48
8	30	44.5	12.00	11.50	2670	396	2.86
9	30	47.6	12.28	11.50	3070	368	3.30
5	30	46.9	12.25	11.50	3420	533	3.33
6	30	47.0	12.00	11.50	3500	392	3.36
7	30	46.4	12.00	11.75	3450	523	3.33
IIB-1	28	46.8	12.50	12.00*	3580	500	3.10
2	28	46.4	12.25	12.00*	3030	408	3.22
3	28	46.6	12.12	12.00*	2850	372	2.86

*Nominal value.

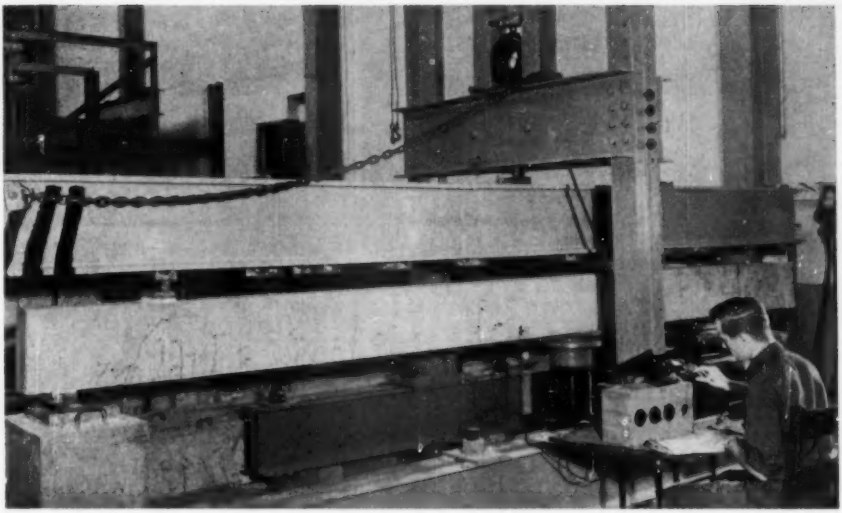

Fig. 2—Typical test setup, Series IIA

capacity of the jack. This beam was retested in a 300,000-lb screw-driven mechanical testing machine.

The beams were supported on concrete pedestals. At the support and load points, the load was transmitted to the beams through a roller of 2-in. diameter and through a bearing block 2 in. thick, 4 in. long, and 6 in. wide. The bearing blocks were bedded in plaster of Paris.

The load was applied in increments of 2 to 10 kips. After each increment of load, deflection and strains were measured. In addition, the cracks were marked on the sides of the beam and photographed. No attempt was made to maintain the load at one level during measurements; before diagonal tension cracking the load decreased only slightly but after diagonal tension cracking the load decreased as much as 1000 lb. Loading was continued until a complete collapse of the beam was reached.

Deflections were measured at the midlength of the beam with a 0.001-in. dial indicator mounted on a deflectometer bearing against the floor of the laboratory.

Strains were measured with bonded-wire electric resistance strain gages. Gages ¼ in. long were mounted on the tension steel at the critical sections and at the midlength of the beam; core holes were provided in the concrete during casting for this purpose. Gages 1 in. long were mounted on the com-

TABLE 4 — AVERAGE DIMENSIONS OF REINFORCING BARS*

Bar size, No.	Area, sq in.	Gap width, in.	Deformation	
			Height, in.	Spacing, in.
3	0.107	0.068	0.025	0.20
7	0.574	0.146	0.052	0.40

*All values are averages for five bars of the same size.

TABLE 5—TEST RESULTS*

Beams	Cracking load, P_c, kips Effective shear spans† 1	2	3	4	Yield load P_y, kips	Ultimate load P_u, kips	Mode of failure‡	Deformation at P_u Strains§ Concrete	Steel	Center deflection, in.
IA-1a		21.8**	—		—	26.6	S	0.00204	0.00067	0.312
b		22.0**	—		—	34.6	S	0.00280	0.00152	0.344
2a	—	39.5	33.4**	—	—	51.1	S	0.00239	0.00127	0.577
b	—	27.8**	—	—	—	27.8	D	0.00200	—	0.250
3a	—	43.2**	—	—	—	49.4	D	0.00134	0.00088	0.335
b	57.4	53.5**	51.2	—	—	57.4	S	0.00500	0.00087	0.430
4a	54.3	61.2**	54.3	61.2	—	61.2	D	0.00088	0.00091	0.373
b	49.3	55.2	55.2**	49.3	—	55.2	D	0.00137	0.00088	0.420
5a	39.1	—	—	49.6**	—	49.6	D	—	0.00080	0.176
b	46.7**	—	—	53.4	—	59.2	S	0.00259	0.00111	0.183
8a	59.2	—	—	55.1**	—	59.2	S	0.00248	0.00093	0.100
b	57.4**	—	—	—	—	68.5	S	0.00240	0.00112	0.120
6a	50.2**	—	—	50.2	—	50.2	D	0.00062	0.00067	0.046
b	61.2	—	—	49.2**	—	79.1	S	0.00215	0.00123	0.165
7a	47.2**	—	—	57.3	88.2	110.5	Y-S	0.00531	0.00325	0.110
b	55.2**	—	—	55.2	—	68.3	S	0.00270	—	0.052
IB-1	—	—	32.2**	—	—	36.8	S	0.00240	0.00095	0.340
2	—	41.2**	—	—	—	55.4	S	0.00220	0.00081	0.358
3	51.3	63.2	—	59.2**	—	63.2	S	0.00185	0.00100	0.130
4	65.5**	—	—	51.3	104.0	121.1	Y-S	0.00284	0.00170	0.190
IIA-1a	—	41.6**			—	41.6	D	0.00092	—	0.295
b	43.0**	56.0			—	63.2	S	0.00512	0.00107	0.250
2	40.4	40.4**			—	55.5	S	0.00380	0.00122	0.568
3	44.4**	44.4			—	50.2	S	0.00354	0.00107	0.400
4a	42.2**	—			—	42.2	D	0.00212	0.00095	0.475
b	47.1**	—			—	47.8	S	0.00290	0.00105	0.090
8	46.4**	—			—	46.4	D	0.00106	0.00050	0.120
9	60.2**	—			—	60.2	D	0.00133	0.00139	0.075
5	64.3**	—			—	64.3	D	0.00120	—	0.080
6	78.1**	—			64.5	78.4	Y-D	0.00435	0.01000	0.115
7	—	—			66.6	77.5	F	0.00852	0.01193	0.600
IIB-1		17.0	20.4**		—	20.4	D	0.00079	0.00122	0.376
2		18.3**	—		—	18.3	D	0.00146	0.00109	0.435
3		—	16.6**		—	16.6	D	0.00069	0.00113	0.454

*All loads include the weight of the load-distributing beams.
†See Fig. 1.
‡S = shear-compression; D = diagonal tension; Y = yielding preceding failure; F = flexure.
§With few exceptions the strains listed are extrapolated values.
**Location of failure.

pressive surface of the concrete adjoining the critical sections and one gage 6 in. long was mounted at midspan on the top surface of the beams in Series I.

TEST RESULTS

Summary of test data

The results of the tests are summarized in Table 5 which includes the diagonal tension cracking loads P_c, the loads at which the reinforcement yielded P_y, the ultimate loads P_u, the modes of failure, and the strains and deflections at ultimate load.

The diagonal tension cracking loads given in Table 5 were determined visually; they are the loads at which diagonal tension cracks were first observed in the given effective shear span. As shown in Fig. 1, Spans 1 and 4 corresponded to the regions of negative moments and Spans 2 and 3 corresponded to the regions of positive moments. In shorter beams the diagonal tension crack usually formed progressively with load, while in longer beams the formation of the crack was sudden. Consequently, the cracking loads could be determined more accurately for long beams.

The load at which the tension reinforcement yielded was determined from the load-strain curves for the reinforcement. The curves showed a definite change of slope at the outset of yielding. The corresponding strain compared well with the yield strain determined from tension tests of coupons.

The ultimate load is the maximum load reached during a test. For most beams it was the collapse load. In some beams, however, the load dropped after reaching a maximum and the beam collapsed as the load was dropping. In still other cases, the load dropped off and then stabilized but subsequent increases in load caused a collapse before the first maximum was reached.

The modes of failure are designated as D for diagonal tension failure, S for shear-compression failure, and F for flexural failure. The letter Y is used to indicate yielding of the reinforcement prior to failure.

For diagonal tension failures, the ultimate load was equal to the highest diagonal tension cracking load (Table 5). In ten beams, failure occurred simultaneously with formation of the first diagonal tension crack. In five beams (IA-3a, 4a, 4b, 5a, and IIB-1), failure occurred with the formation of a diagonal tension crack at a load higher than the *initial* diagonal tension cracking load. With the exception of Beam IA-3a, the crack causing failure formed in another effective shear span; in Beam IA-3a the crack causing failure formed in the effective shear span which already contained a diagonal tension crack. In one beam (IIA-6) yielding of the reinforcement occurred before the formation of the diagonal tension crack but the failure resembled a diagonal tension failure; this failure is designated as Y-D.

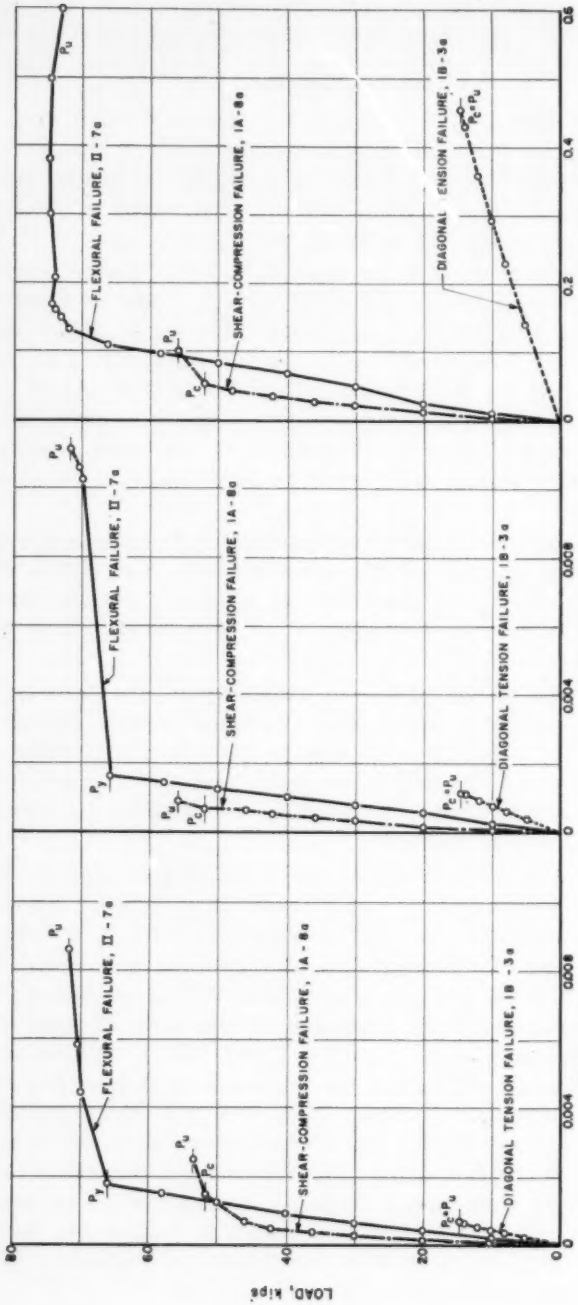

Fig. 3—Typical load-deformation curves

For shear-compression failures, the ultimate load was greater than the diagonal tension cracking loads. In two beams (IA-7a and IB-4) the steel yielded after the formation of the diagonal tension crack, but the failures resembled shear-compression failures and are designated as Y-S.

The fourth beam in which yielding occurred (IIA-7) failed in flexure.

The steel and concrete strains given in Table 5 were obtained from measurements at the section of failure. It was seldom possible to measure the strains at the maximum load; therefore extrapolated values are listed for most beams.

The deflections were measured at the midlength of beams. For Series I, the midlength coincided with midspan; for Series II, the midlength coincided with the section of zero moment. In most tests, the deflection was observed at the maximum load; otherwise the value given in Table 5 was obtained by extrapolating the load-deflection curve.

Response of beams to loading

During several initial increments of load, the behavior of all beams was characteristic of reinforced concrete flexural members. Flexural tension cracking occurred first in the regions of maximum moment; it was accompanied by a change in the slope of the load-deformation curves (Fig. 3). With increasing loading the tension cracking spread into the effective shear spans and some cracks bent in the direction of increasing moment. This change in the direction of tension cracks was believed to be the result of the presence of shear; nevertheless, shear appeared to have no practically significant effect on the over-all beam behavior during the initial phase of loading.

The flexural phase of behavior ended with the formation of the initial diagonal tension crack (load P_c in Fig. 3). Without exception, this first diagonal tension crack formed in an effective shear span containing the maximum moment (Table 5). In beams with the moment ratio equal to one, it formed always in the effective shear span with the smaller effective depth of the tension reinforcement. In six beams (IA-4a, 4b, 6a, 7b, IIA-2, 3) the initial diagonal tension cracks formed simultaneously in two effective shear spans subjected to moments and shears of equal magnitude.

Each initial diagonal tension crack was located in one effective shear span some distance away from the point of contraflexure.* This is illustrated in Fig. 4 in which the distance a_c between the point of contraflexure and the point of intersection of the initial diagonal tension crack with the tension reinforcement is plotted as a function of the

*In Beam IIA-1a the diagonal tension crack intersected the tension reinforcement at the point of contraflexure. The crack passed through a core hole formed in the side of the beam at the point of contraflexure for the attachment of a strain gage to the tension reinforcement. Since the effective shear span was short, it was suspected that the presence of this core hole may have precipitated the formation of the crack. The companion specimen, IIA-1b, was cast without a core hole at the point of contraflexure; in this beam the diagonal tension crack did not pass through the point of contraflexure.

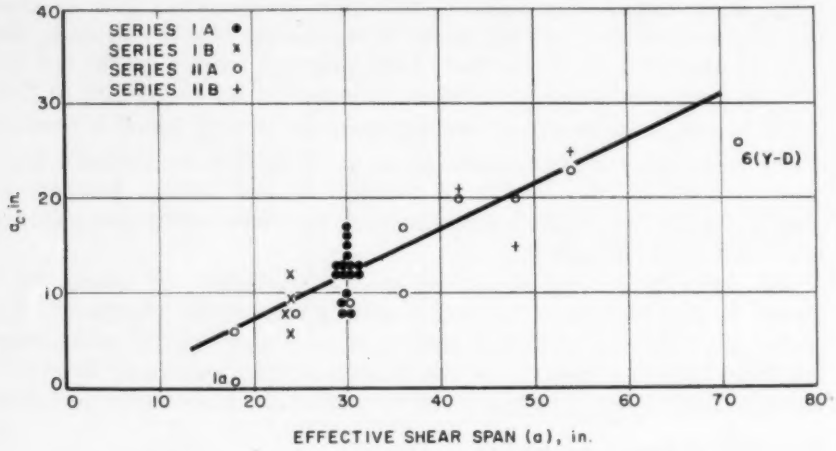

Fig. 4—Location of initial diagonal tension cracks

length of the effective shear span. The crack extended from the point of intersection with the tension reinforcement toward the section of maximum moment in the same effective shear span as can be seen in Fig. 5a and 5b.

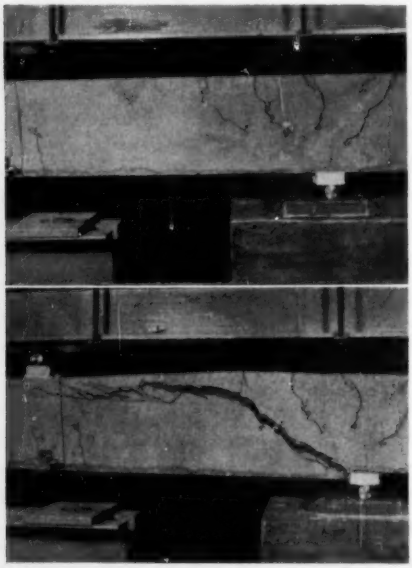

Fig. 5a—Typical crack patterns: diagonal tension failure, Beam 1A-5a. Top: $P = 45$ kips. Bottom: After failure ($P_u = 49.6$ kips)

The exact locations of the sections at which the initial diagonal tension cracks began to form are not known. It was observed, however, that the cracks often coincided with the bent portion of one of the tension cracks formed prior to diagonal tension cracking and located between the section of maximum moment and the section at which the diagonal tension crack intersected the tension reinforcement.

In ten beams the formation of the initial diagonal tension crack was accompanied by a complete collapse. The diagonal tension crack extended to the compression surface at one end and split the beam along the tension reinforcement at the other end. The failures occurred suddenly without warning at relatively low maximum strains

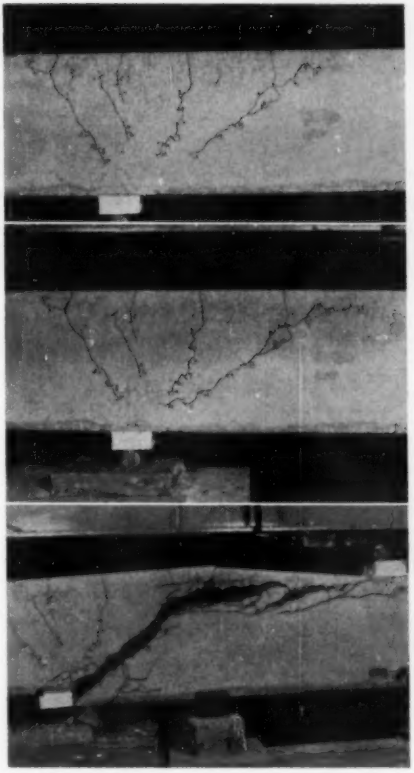

Fig. 5b—Typical crack patterns: shear-compression failure, Beam IA-8b. Top: $P = 55.3$ kips. Middle: $P = 60.2$ kips. Bottom: After failure ($P_u = 68.5$ kips)

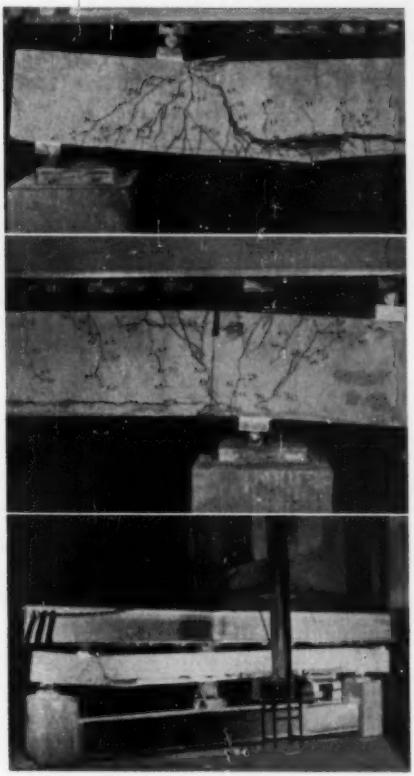

Fig. 5c—Typical crack patterns: Beam IIA-7a after flexural failure. Top: Positive moment region. Middle: Negative moment region. Bottom: Over-all view

in both concrete and steel (Fig. 3 and Table 5). They are designated as diagonal tension failures.

In 23 beams the initial diagonal tension crack terminated some distance short of the compression surface and further increases in load were needed to reach failure. The initial diagonal tension cracking resulted in a decrease of stiffness manifested by the increased rate of deflection and concrete strain (Fig. 3). In five beams (IA-3a, 4a, 4b, 5a, and IIB-1) one or more diagonal tension cracks formed at higher loads and failure occurred simultaneously with the formation of one of these new cracks (Fig. 5a). The failure was again sudden although the formation of the preceding diagonal tension cracks indicated distress prior to the actual collapse. Failures of these beams are also designated as diagonal tension failures.

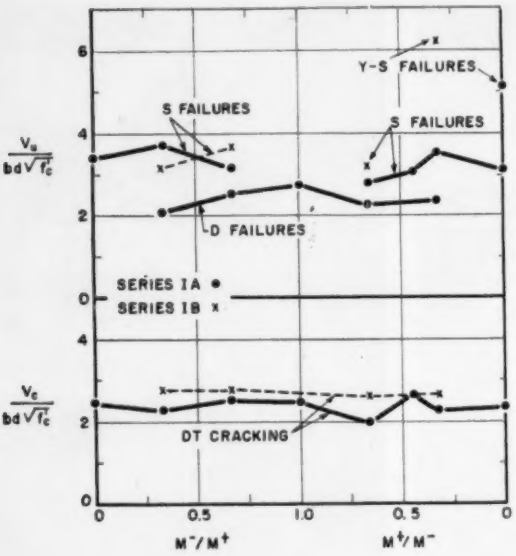

Fig. 6—Effects of moment ratio

The remaining 18 beams with diagonal tension cracks failed by destruction of the compression zone of concrete at the critical section at the compression end of one of the existing diagonal tension cracks. These failures are designated as shear-compression failures (Fig. 5b). It may be noted in Table 5 that only four beams (IA-3b, IIA-1b, 2, and 3) had two diagonal tension cracks in the shear span in which failure occurred. However, even in these four beams no signs of bond failure were noted along the compression reinforcement.*

Shear-compression failures were sudden, although less abrupt than the diagonal tension failures. They were accompanied by high maximum compressive strains in concrete (an average of 0.00308) but by relatively low steel strains; with two exceptions (IA-7a, IB-4), the steel strain was always below the yield point value.

Beam IIA-7 failed in flexure by extensive yielding of the tensile reinforcement. The yielding was followed by crushing of concrete at both critical sections and by splitting along the bottom reinforcement the full length between the critical sections (Fig. 5c). The deflections, concrete strains, and steel strains, shown in Fig. 3, illustrate the ductile behavior of this beam. It may be noted that no diagonal tension cracks formed in this beam and that the maximum steel strain entered the strain hardening region.

*In an earlier study of restrained beams[1] bond failures along the compression reinforcement were common; most of the specimens had very short shear spans.

Findings of Series I

The principal variable of Series I was the moment ratio. In addition, minor variations were encountered in the strength of concrete and in the effective depth of reinforcement.

The effects of the moment ratio on the magnitude of shear at initial diagonal tension cracking are illustrated in the lower portion of Fig. 6.* The dots connected by the full line represent averages for two companion specimens of Series IA. The crosses connected by the dashed line represent individual specimens of Series IB. The data for both series indicate that the moment ratio has no effect on the magnitude of shear at initial diagonal tension cracking.

The variations in the moment ratio were achieved by varying the length of the shear span and the load ratio P_2/P_1 (Table 1). The variations in the load ratio have been accounted for in Fig. 6 through consideration of the shear rather than the load. Accordingly, the variations in the moment ratio may be considered the result of the variations in the length of the shear span. Apparently, *if all other properties are kept constant, the length of the shear span, a', has no effect on the magnitude of shear at initial diagonal tension cracking.* This point is further illustrated in Fig. 7. It should be noted, however, that while the length of the shear span was variable, the moment-shear ratio was constant both in Series IA and in Series IB.

The effect of the moment ratio on the magnitude of shear at the maximum load may be examined in the upper portion of Fig. 6. The dots represent average data for companion specimens that failed by the same mode, but individual data wherever the companion specimens failed by different modes (two dots for the same moment ratio). It will

*To compensate for minor variations in d and f_c' in Fig. 6, 7, and 9, the shear forces were divided by the quantity $bd\sqrt{f_c'}$.

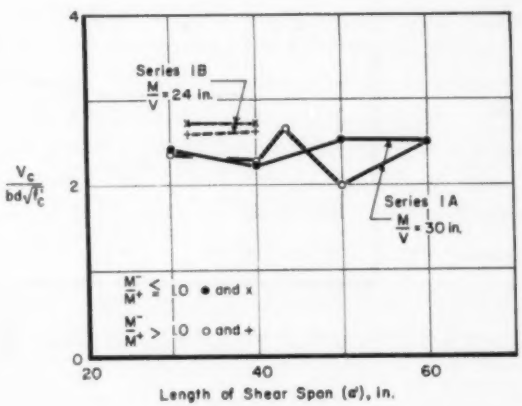

Fig 7—Effect of length of shear span on diagonal tension cracking loads

be noted that in only three out of eight sets of companion specimens the failure occurred in the same mode. Furthermore, the differences in the failure loads of the companion specimens were substantial, the maximum being ± 29.1 percent from the average (Table 5). Finally, Beams Ia-7b and IB-4 failed in shear compression after yielding of the tension reinforcement at loads very much in excess of those that

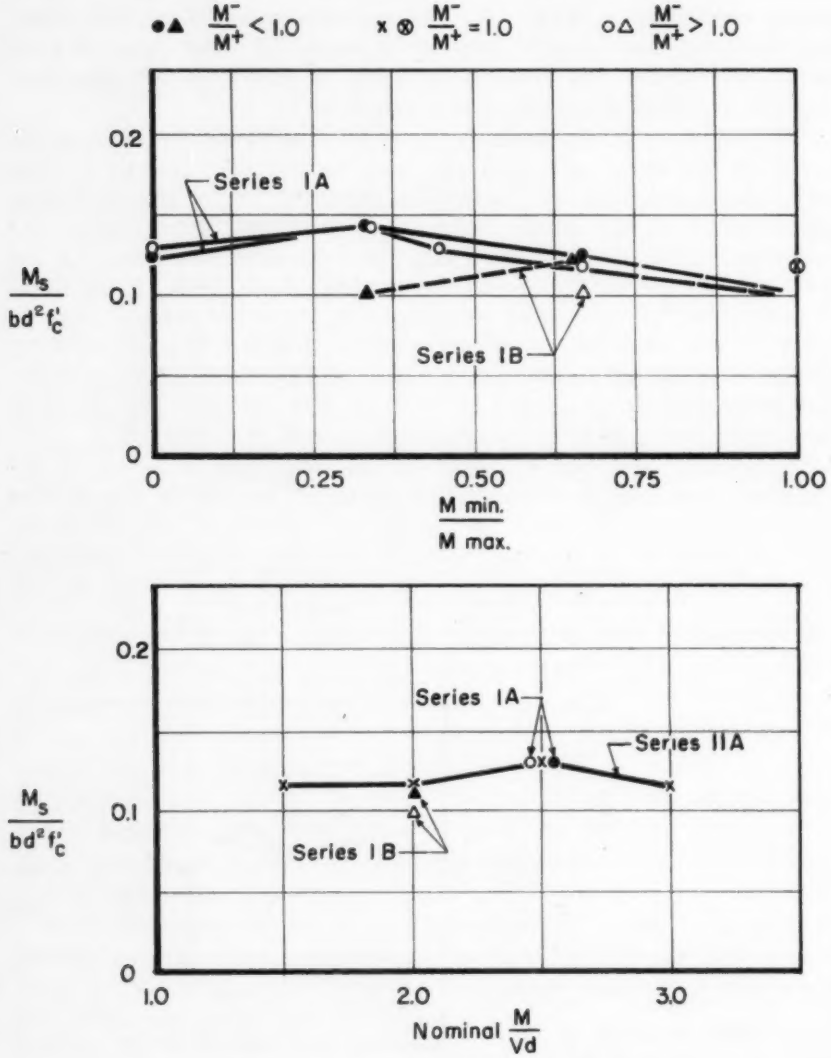

Fig 8—Variations in shear-moment capacity

caused failure of other specimens. No conclusive explanation was found for these differences but it is believed that they were caused by random differences in the paths of the diagonal tension cracks. An analogous finding was reported by Hanson[2] for simply supported beams. Thus, it appears that for beams of the type studied in Series I the excess strength beyond diagonal tension cracking is unreliable. The data are inconclusive concerning the effect of the moment ratio on the shear strength.

The effect of the moment ratio on the shear-moment capacity is illustrated in the upper portion of Fig. 8.* No definite trends with the moment ratio can be noted particularly because of the differences between the results of Series IA and IB. However, the relatively low strength of Beams IA-4a and 4b (circled cross in Fig. 8), both of which failed in diagonal tension, suggests a possible decrease of the shear-moment capacity as the ratio M_{min}/M_{max} approaches unity.† In view of the apparent lack of bond failures along the compression reinforcement, this indicates that the shear-moment capacity may be a function of the length of the shear span. A similar finding was reported earlier for simple beams and knee frames.[3-5]

Findings of Series II

The principal variable of Series II was the moment-shear ratio. The effects of the moment-shear ratio on the magnitude of shear at initial diagonal tension cracking and at failure are illustrated in Fig. 9. The figure illustrates also the effect of M/Vd on the mode of failure.

The shear, divided by $bd\sqrt{f_c'}$, is plotted as a function of the moment-shear ratio, M/Vd, for each beam of Series IIA and IIB. The heavy line represents failures; the light line represents diagonal tension cracking without failure. It will be noted that beams with M/Vd less that 3.2 failed in shear compression; beams with M/Vd larger than 3.2 but smaller than 6.3 failed in diagonal tension, and the one beam with M/Vd in excess of 6.3 failed in flexure.‡ Analogous findings were reported earlier for simply supported stub beams and knee frames[3] and for simply supported prismatic beams.[6]

The variations in the M/Vd were obtained by varying the load ratio and the length of the shear span. For Series IIA, the effective shear span, a, was equal to one half of the length of the shear span, a', while

*To compensate for minor variations in d and f_c', the shear moments M_s were divided by the quantity bd^2f_c'.

†Fig. 8 does not include data for Specimens IA-7a and IB-4 which failed in shear-compression after yielding of the longitudinal reinforcement. Inclusion of the data for these two beams also suggests a decreasing trend with M_{min}/M_{max}.

‡The only exceptions to this pattern of behavior were two beams which failed in diagonal tension but had M/Vd smaller than 3.2. One of these, Beam IIA-4a ($M/Vd = 3.0$) suggests that the erratic behavior in Series IA could be characteristic of beams with relatively small margin of load capacity beyond the initial diagonal tension cracking load. The second beam, IIA-1a, exhibited an erratic behavior not only as to the mode of failure but also as to the location of the diagonal tension crack (see the discussion of the response of beams to loading.)

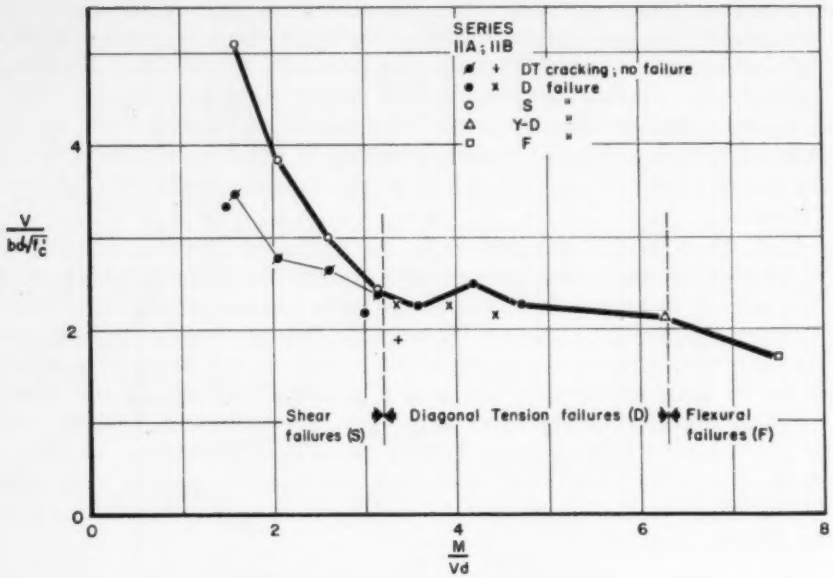

Fig. 9—Effect of moment-shear ratio on diagonal tension cracking and failure

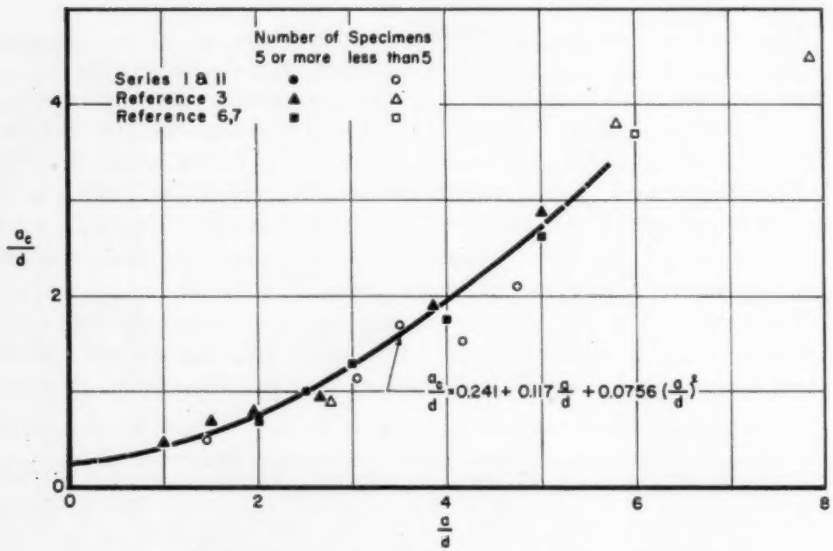

Fig. 10—Location of a diagonal tension crack

for Series IIB the two quantities were equal. Thus, if a'/d were substituted for M/Vd in Fig. 9, the position of the points representing beams of Series IIB would remain unchanged while the horizontal ordinates of all points representing beams of Series IIA would be doubled. Obviously, the crosses would fall considerably below the line representing diagonal tension cracking in Series IIA and considerably to the left of the limit between the shear-compression and diagonal tension failures in Series IIA. This lack of correlation between the two series on the basis of the length of the shear span corroborates the finding of Series I that, by itself, the length of the shear span has no significant effect on the diagonal tension cracking loads.

The effect of M/Vd on the shear-moment capacity of beams of Series IIA is illustrated in the lower portion of Fig. 8. Averages for Series IA and IB are included for comparison. Within the range of the data of this investigation, the shear-moment capacity appears to be independent of M/Vd, that is, independent of the length of the *effective* shear span.

ANALYTICAL STUDIES

Location of a diagonal tension crack

It has been shown in Fig. 4 that the distance a_c between the intersection of the initial diagonal tension crack with the tension reinforcement and the section of zero moment increased with increasing length of the effective shear span. Similar findings were reported earlier for simply supported prismatic beams[6,7] and for stub beams and knee frames.[3] The data from all three investigations are plotted in Fig. 10.

Fig. 10 includes only averages for specimens of the same investigation having the same a/d. Each solid symbol represents five or more specimens; each open symbol represents less than five specimens.

The line fitting best the trend indicated by the solid symbols* gives the following relationship between a_c/d and a/d:

$$\frac{a_c}{d} = 0.241 + 0.117 \frac{a}{d} + 0.0756 \left(\frac{a}{d} \right)^2 \quad \text{.....(1)}$$

An examination of Fig. 4 reveals the magnitude of deviation which may be expected for individual specimens from this average line.

Initial diagonal tension cracking

Earlier studies[3,4,6-8] have shown that the magnitude of shear at initial diagonal tension cracking is a function of the strength of concrete, percentage of tension reinforcement, and beam dimensions, in addition to the ratio of moment to shear. It has been suggested

*The line was fitted by the method of least squares.

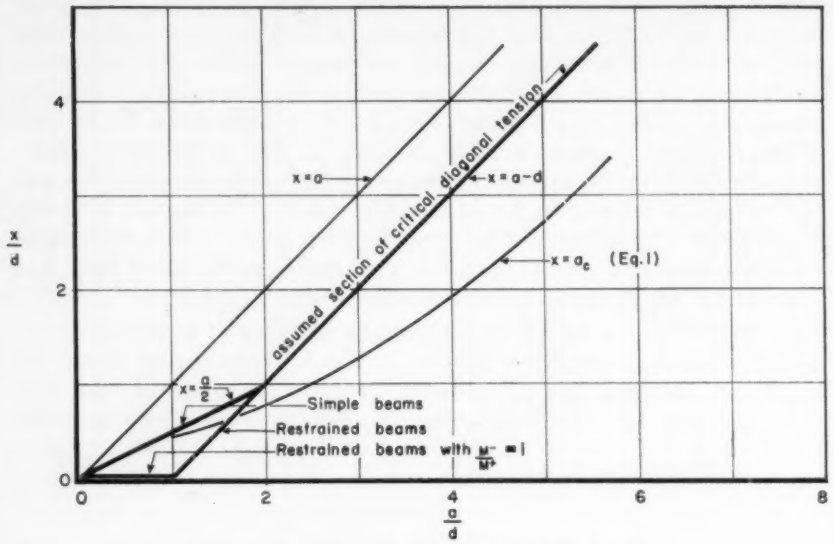

Fig. 11—Section of critical diagonal tension

that the external shear may be correlated with the section properties on the basis of the following expression:[9*]

$$\frac{V_c}{bd\sqrt{f_c'}} = A + B\left(\frac{V}{M}\right)_c \frac{pd}{\sqrt{f_c'}} \qquad (2)$$

where A and B = empirical constants

$(V/M)_c$ = ratio of shear to moment at the section at which the diagonal tension crack begins to form

Before a correlation on the basis of Eq. (2) can be made it is necessary to assume the location of the section at which the initial diagonal tension crack begins to form. Although no direct data are available on the location of this section, it is obvious that it must be somewhere between the section of maximum moment and the section at which the crack intersects the tension reinforcement. The locations of the latter two sections are shown in Fig. 11 as lines $x = a$ and $x = a_c$.

For simplicity, it was assumed that the section of initial diagonal tension cracking in a shear span is located at the distance d from the section of maximum moment, but no closer than half-way to the other

*From consideration of the principal stress at a point, Morrow[10] suggested the grouping of the external shear and of the section properties into two dimensionless parameters:

$$V_c/bdf_t' \text{ and } (V/M)_c npd$$

The parameters used in Eq. (2) were obtained from Morrow's parameters by expressing the resistance of concrete, f_t', to the principal tensile stress as a direct function of $\sqrt{f_c'}$ and the modular ratio, n, as a direct function of $1/\sqrt{f_c'}$.

end of the shear span. For such location, indicated in Fig. 11 by the heavy lines, the ratio of moment to shear may be expressed as

$$\left(\frac{M}{V}\right)_c = a - d \geq a - 0.5a' \quad \text{.....(3)}$$

The two parameters in Eq. (2) were evaluated for every specimen on the basis of the lowest diagonal tension cracking loads (Table 5) and plotted in Fig. 12. A regression analysis, utilizing the method of least squares, gave the following equation for the shear at initial diagonal tension cracking:

$$\frac{V_c}{bd\sqrt{f_c'}} = 1.917 + 2725\left(\frac{V}{M}\right)_c \frac{pd}{\sqrt{f_c'}} \quad \text{.....(4)}$$

where V_c = external shear at diagonal tension cracking, lb

b,d = dimensions of the cross section, in.

p = ratio of tension reinforcement

$(M/V)_c$ = ratio of moment to shear given by Eq. (3), in.

f_c' = compressive cylinder strength of concrete, psi

It may be noted that, as the horizontal ordinate in Fig. 12 approaches zero, the line represented by Eq. (4) approaches infinity; obviously, an upper limit would be needed for general applicability of Eq. (4). However, no such limit is necessary within the range of this investigation.

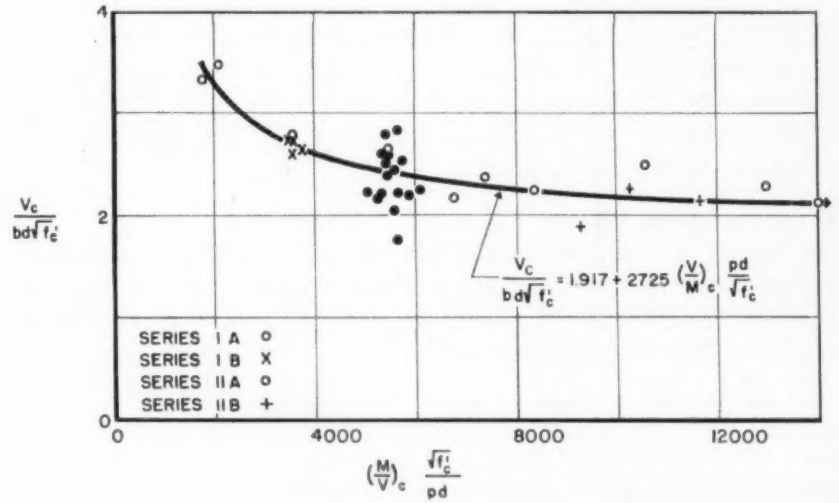

Fig. 12—Shear at initial diagonal tension cracking

TABLE 6 — NOMINAL SHEARING STRESS AT DIAGONAL TENSION CRACKING

Beam	$v_c$test,* psi	$v_c$calc, psi	$\frac{v_c test}{v_c calc}$	Beam	$v_c$test,* psi	$v_c$calc, psi	$\frac{v_c test^*}{v_c calc}$
IA–1a	151	139	1.08	IIA–1a	191	199	0.96
b	148	156	0.95	b	210	195	1.08
2a	146	144	1.01	2	156	150	1.04
b	122	143	0.85	3	149	136	1.09
3a	139	151	0.92	4a	117	125	0.94
b	168	143	1.18	b	137	132	1.04
4a	149	139	1.07	8	116	116	1.00
b	134	136	0.98	9	138	121	1.15
5a	104	141	0.73	5	133	124	1.07
b	123	136	0.90	6	126	122	1.03
8a	150	142	1.06				
b	156	135	1.15	IIB–1	113	132	0.86
6a	131	136	0.96	2	124	120	1.04
b	131	142	0.92	3	114	115	0.99
7a	123	135	0.91		Average		1.022
b	144	139	1.04				
IB–1	155	152	1.02				
2	151	150	1.01		Average Series I and II		1.000
3	150	155	0.97				
4	156	156	1.00				
	Average		0.986		Standard deviation		0.091

*Based on initial diagonal tension cracking loads.

The nominal shearing stress at initial diagonal tension cracking, $v_c = V_c/bd$, computed from Eq. (2), is compared with the corresponding test values in Table 6. It will be noted that the average ratio of $v_c test/v_c calc$ for all specimens is equal to 1.000 and the standard deviation for this ratio is 0.091.

Data for all specimens of Series IA, shown in Fig. 12 as dots, fall within a narrow range of the horizontal ordinate. Thus, they permit an estimate of the natural scatter associated with diagonal tension cracking. For the 16 specimens of Series IA, the standard deviation for the parameter $V_c/bd\sqrt{f_c'}$ was equal to 0.269. The standard error of estimate by Eq. (2) was 0.272.

Shear-compression failures

In view of the apparent lack of bond failures along the compression reinforcement, the effect of the compression reinforcement may be neglected[8] and the moment equilibrium at the critical section of failure may be written as for simple beams:[3]

$$\frac{M_s}{bd^2 f_c'} = \frac{pf_s}{f_c'}\left(1 - 0.44\,\frac{pf_s}{f_c'}\right) \qquad (5)$$

where M_s = moment at shear-compression failure

f_s = steel stress at shear-compression failure

Assuming that the strain distribution at critical section includes a concentrated angle change at the neutral axis, the steel stress at failure may be related to the ultimate strain in concrete, ε_u, and the section properties as follows:[3]

$$f_s = \tfrac{1}{2} E_s K\varepsilon_u \left(-1 + \sqrt{1 + \frac{4k_1k_3f_c'}{pE_sK\varepsilon_u}} \right) \quad \text{.....(6)}$$

where E_s is the modulus of elasticity of tension reinforcement, parameters k_1k_3 define the magnitude of the compressive force resisted by concrete at the critical section, and parameter K is a measure of the magnitude of the concentrated angle change.

Earlier studies[3] suggested that k_1k_3 is a function of the strength of concrete:

$$k_1k_3 = \frac{800 + f_c'}{70 + f_c'} \quad \text{.....(7)}$$

where the strength of concrete, f_c', is expressed in psi. As this investigation included only minor variations in concrete strength, Eq. (7) was adopted for this study.

Assuming that the concentrated angle change is caused by opening of the diagonal tension crack, it can be shown that parameter K is a function of the ratio of the length of the diagonal tension crack to the depth of the beam below the neutral axis, the inclination of the diagonal tension crack, the bond characteristics of the tension reinforcement, and the distribution of the compressive stresses in concrete along the length of the beam. Assuming further that the first two items are functions of the ratio of the length of the shear span, a', to the beam depth, d, and that the last two items were essentially constant for this series of tests, the quantity $K\varepsilon_u$ was evaluated from the moment at shear failure with the aid of Eq. (5), (6), and (7). The following expression was then obtained by the method of least squares:

$$10^4 K\varepsilon_u = 0.953 + 0.62 \frac{d}{a'} \quad \text{.....(8)}$$

The accuracy of Eq. (5) through (8) may be judged from the values of moment at shear failure given in Table 7. It will be noted that the

TABLE 7 — MOMENT AT SHEAR COMPRESSION FAILURE

Beam	$M_s$test,* in.-kips	$M_s$calc,† in.-kips	$\frac{M_s\text{test}}{M_s\text{calc}}$
IA–1a	372	368	1.01
1b	484	438	1.11
2a	460	350	1.31
3b	381	364	1.05
5b	311	335	0.93
8a	311	340	0.91
8b	360	322	1.12
6b	415	355	1.17
7b	359	362	0.99
IB–1	280	359	0.78
2	323	345	0.94
3	298	369	0.81
IIA–1b	337	348	0.97
2	333	356	0.94
3	325	317	1.02
4b	325	322	1.01
		Average	1.004
		Standard deviation	0.128
IA–7a	578	576‡	0.99
IB–4	571	542‡	1.05

*At the critical section of failure.
†Calculated from Eq. (5), (6), (7), and (8) except for Beams IA–7a and I–B4.
‡Calculated from Eq. (5) with f_s equal to the yield point stress.

TABLE 8 — MOMENT AT FLEXURAL FAILURE

Beam	Stage of behavior	Concrete strain	M_f test, in.-kips	M_f calc, in.-kips	M_f test, M_f calc,
II-7a	First crushing	0.00575	595	587	1.01
	Final failure	0.01193	634	627	1.01

average ratio $M_s test/M_s calc$ is 1.004 and the corresponding standard deviation is 0.128. In view of the differences in the capacities and modes of failures of the companion specimens, this relatively large standard deviation does not appear excessive.

Table 7 includes also Specimens IA-7a and IB-4 which failed in shear-compression after yielding of the tension reinforcement. Their shear-moment capacities computed from Eq. (5) on the basis of the yield point stress are in good agreement with the test data. However, computations based on Eq. (6) through (8), rather than on the yield point stress, suggest that the beams should have failed in shear-compression at a lower load.

Flexural failure

Beam IIA-7 failed in flexure without formation of a diagonal tension crack. Before the failure load was reached, concrete began to crush and the stresses in the tension steel entered the strain hardening range. However, it can be seen in Fig. 3b that the extent of strain hardening was small.

The flexural moment capacity of Beam IIA-7 was computed for two stages of loading, at first crushing of concrete and at failure, from equations given in Reference 11. The calculations for first crushing were based on the yield point stress of the steel; an estimated maximum stress, including the effect of strain hardening, was used in computing the moment at failure.

The test data and the computed moment capacities are given in Table 8. It can be seen that the agreement between the corresponding test data and calculated values is excellent.

SUMMARY

The objectives of this investigation were to determine the effects of a varying moment ratio and a varying moment-shear ratio on the shear strength of restrained reinforced concrete beams. A total of 34 rectangular beams were tested with concentrated loads in two series. Series I had the moment ratio and Series II had the moment-shear ratio as the major variable. The nominal concrete strength, steel percentage, and beam cross section were held constant throughout both series. The longitudinal reinforcement extended the full length of the

beams at both the top and the bottom; there was no web reinforcement in the portions of the beams in which failure occurred.

Fifteen beams failed upon formation of a diagonal tension crack, while 18 beams sustained additional load after formation of the diagonal tension cracks and eventually failed in shear-compression. One beam failed in flexure without formation of any diagonal tension cracks.

Analyses of the diagonal tension cracking loads have shown that the length of the shear span and the moment ratio have no direct effect on the diagonal tension cracking load. On the other hand, an increasing moment-shear ratio has the effect of decreasing the load at which the diagonal tension crack forms. The load causing diagonal tension cracking may be computed on the basis of a limiting nominal shearing stress. An equation, based on the results of these tests, is presented. The equation is applicable to both simple and restrained beams.

There is evidence from Series IA that the path followed by the diagonal tension crack may influence the mode of failure. In four pairs of companion beams, one beam failed by shear-compression while the other beam failed by diagonal tension. It seems probable that for beams without web reinforcement, the strength beyond the diagonal tension cracking load depends on the chance location of the diagonal tension crack. Therefore, the diagonal tension cracking load should be considered the ultimate design load for beams without web reinforcement.

The tests have shown that the shear-moment capacity of reinforced concrete beams without web reinforcement is unreliable and, therefore, of academic interest only. Although the experimental evidence presented is inconclusive, it suggests that the shear-moment capacity of beams without web reinforcement is independent of the moment-shear ratio but may decrease slightly with increasing length of the shear span. A set of equations, based on equilibrium and compatibility conditions, is presented for the shear-moment capacity of both restrained and simple beams without web reinforcement.

The observed moments at first crushing of concrete and at failure of the beam failing in flexure were in excellent agreement with the equations of an earlier investigation.[11]

ACKNOWLEDGMENT

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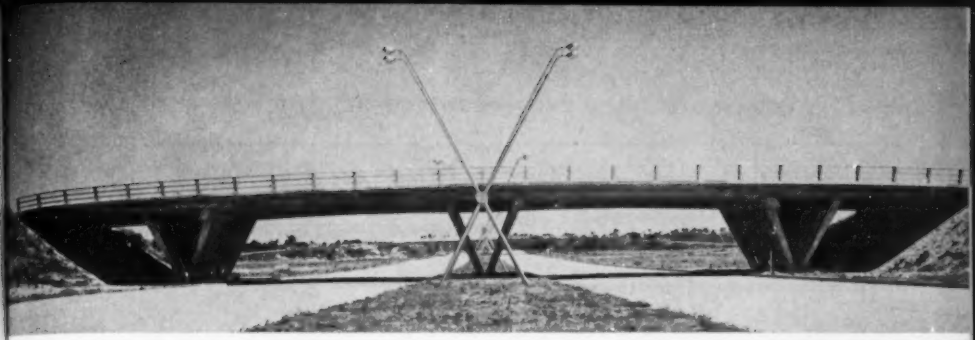

Title No. 57-5

Reinforced Concrete Slab Bridges for the Via Monumental, Havana, Cuba

By LUIS P. SÁENZ and IGNACIO MARTÍN

Design and construction features are described for a series of reinforced concrete slab highway bridges with inclined columns and 59, 62, and 79 ft maximum spans. The economies and the advantages of slab deck bridges for overhead crossings and underpasses are discussed.

■ THE DEVELOPMENT OF A NEW CITY, Habana del Este, on the east side of Havana Bay has resulted in the erection of many modern facilities. Construction of a four-lane vehicular tunnel under the entrance channel of Havana Bay was begun in 1956. A related major project for the new city was the Via Monumental (Monumental Highway), a wide avenue providing rapid access from the City of Havana and which continues on to join the road which completes the tourist route leading to Varadero Beach.

Via Monumental is an avenue consisting of two independent highways having three rapid transit lanes each, with a width of 36 ft, separated by a 26-ft mall. Both overhead crossings and underpasses were used in the various grade separations. One clover-leaf intersection, utilizing two overpasses, gives access to five urban developments.

The promoters who planned Habana del Este wanted all the bridges of the overhead and undergrade crossings on Via Monumental to symbolize the modern construction which is expected to develop in the vicinity of the highway. With the sea as a background, the reinforced concrete bridges were designed with thin slabs and inclined columns to give the shape and lightness required to resemble the flight of sea gulls, which form part of the scenery observed from Via Monumental.

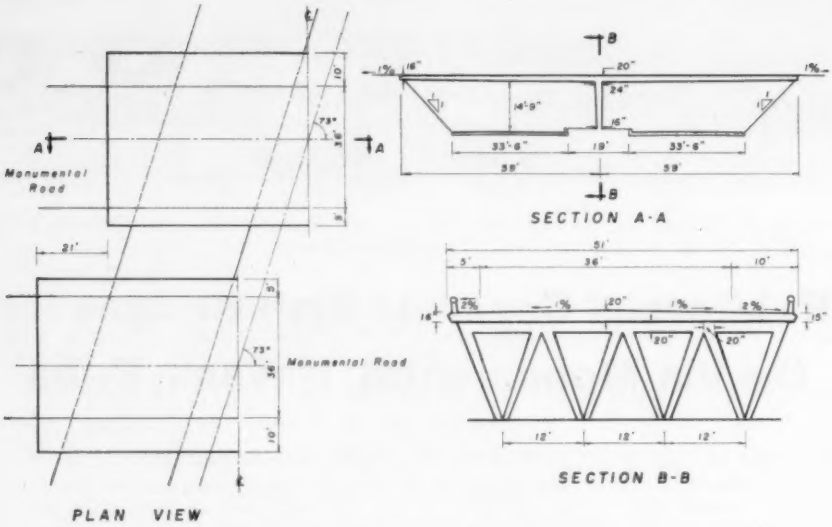

Fig. 1—Skew underpass

Five cases of grade separation were considered: two underpasses and three overhead crossings, two of which were identical and were used in the clover-leaf intersection.

UNDERGRADE CROSSINGS

The underpasses consist of two independent bridges with three lanes, each with a width of 12 ft and two shoulders 5 and 10 ft wide. Reinforced concrete railings are provided. In one of the undergrade crossings the roads meet with a skew of 73 deg, and this was solved by displacing the bridges longitudinally 21 ft (Fig. 1).

Each of these undergrade crossings has two 59-ft spans. The central support is formed by columns inclined transversely to the bridge, but vertical in the longitudinal direction of the bridge (Fig. 2), since a wide

Fig. 2—Standard underpass

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central strip was not available in the streets of the urban developments. The transverse inclination was given to maintain unity in the design of the bridges and obtain effective transverse bracing.

OVERHEAD CROSSINGS

One of the overhead crossings (Fig. 3) consists of a single four-lane bridge, with lanes 12 ft wide and two 4½-ft sidewalks, due to the fact that this bridge forms part of one of the streets of an urban development adjoining Via Monumental. This overhead crossing has a skew of 65 deg to solve the 77 deg angle of crossing of the two roads.

This overhead crossing has two 79-ft spans, but the central support (Fig. 4) is formed by two groups of four columns, inclined longitudinally as well as transversely to the axis of the bridge. These columns meet at a point on the pedestal of the foundation common to the four columns. With this type of support the clear span is reduced to 72.5 ft, creating a central span of 13 ft between the columns, which is a favorable solution in spite of the fact that the inclined columns produce tension in the central span. In addition the foundation was simplified, natural bracing was obtained in two perpendicular directions, visual obstruction was

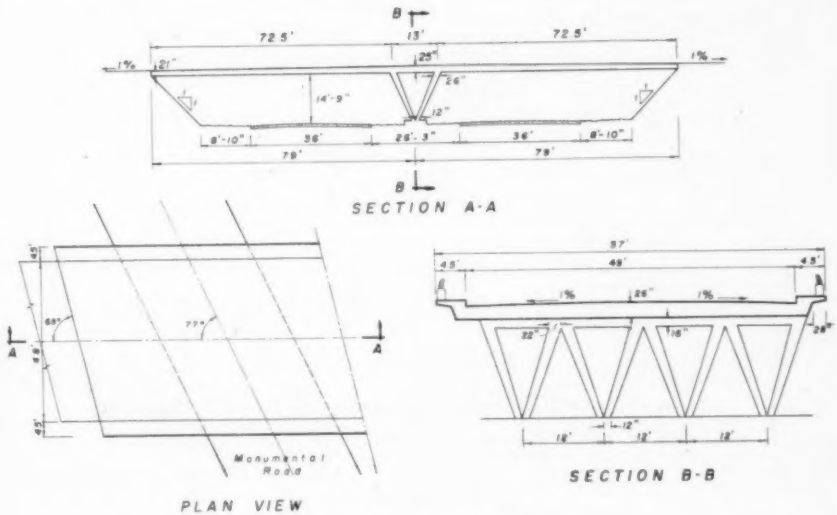

Fig. 3—Overhead crossing

Fig. 4—Central support of overhead crossing

reduced for the drivers of vehicles, and a certain degree of beauty was achieved.

The bridges of the overhead crossings in the clover-leaf are curved. Each one has four lanes, each 10 ft wide, and two 5-ft sidewalks.

Both bridges of the clover-leaf intersection (Fig. 5) have four spans: two 39-ft end spans and two 62-ft central spans. The columns are inclined in the direction of the bridge axis as well as transversely to it, and the axes of the columns coincide in a point of the pedestal on the common foundation. This type of support, having the same advantages as the support of the overhead crossing described above, produces a continuous slab with seven symmetrical spans.

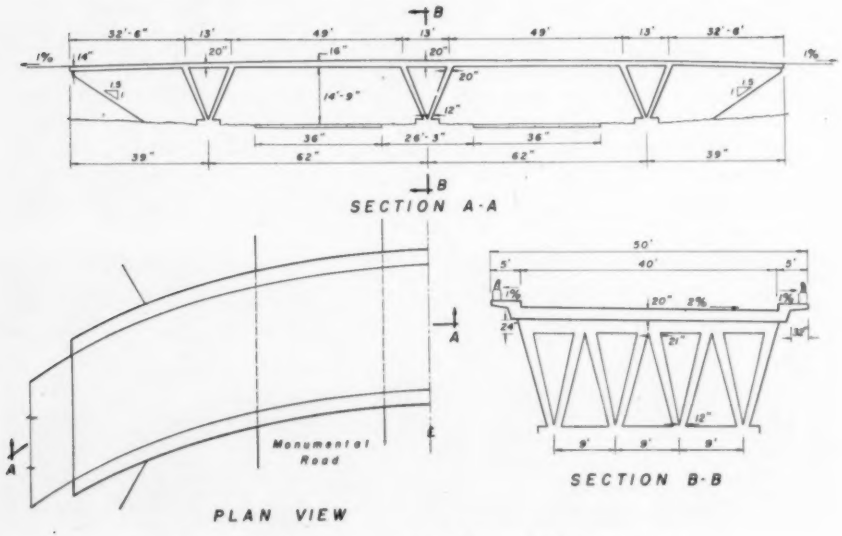

Fig. 5—Overhead crossing in clover-leaf intersection

DESIGN FEATURES

The intrados of the slab for all the bridges was designed flat and horizontal to maintain a uniform vertical clearance of 14 ft 9 in. throughout the structure. However, the depth of the slab was increased over the central supports, which increased the strength of the slab in the region of maximum negative moments, providing at the same time a 1 percent grade toward the bridge abutments and reducing the height of the approach fills.

The bridges were designed for AASHO H-20 loading, as well as a sidewalk load of 60 psf. The ultimate strength method was used in the design, employing a factor of safety of 1.33 for the dead load and 2.00 for the live load.

The slab was designed as a continuous beam over the columns. The final moments in the slab at the interior columns as compared with the moments at the center of a simply supported slab with the same span were reduced about 15 percent.

The inclined columns induced an axial tension in the interior slabs and reinforcing bars were added to resist that tension. Bending moments in the inclined columns were relatively small.

Special care was taken to give the bridge sufficient camber to provide for the elastic and the plastic deformations of the reinforced concrete.

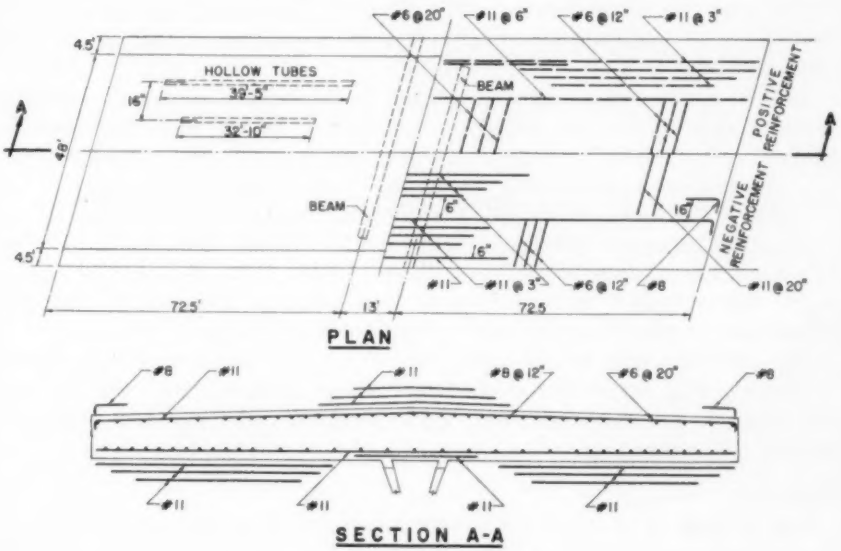

Fig. 6—Slab reinforcement

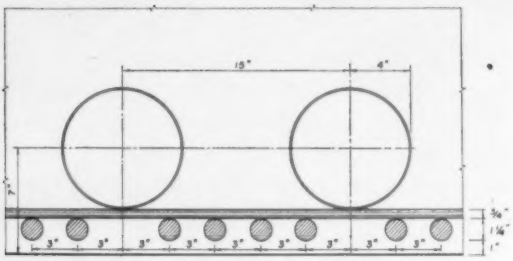

Fig. 7—Slab section, positive reinforcement

Abutments

The concealed abutments consist of column bents with maximum rigidity in the direction of the bridge axis, having taken into account the possibility of longitudinal thrust by displacement of the rock fill approach.

Materials

All the concrete for the bridge slabs was required to be 4000 psi at 28 days. Although a lower value would have been adequate for bond, flexure, and shear stresses, this quality of concrete was used to furnish adequate protection for the steel reinforcement due to the proximity of the sea and to diminish the deflection of the bridges.

The main reinforcement of these bridges consists of deformed bars 1 and 1¼ in. in diameter; temperature reinforcement is deformed bars ¾ in. in diameter. The positive reinforcement is a network independent of the negative reinforcement (Fig. 6 and 7), obviating the necessity of bent bars, to facilitate placement of steel, accelerate construction, and reduce the labor cost.

Hollow cardboard tubes were placed in the slab to reduce its weight, without appreciably diminishing the rigidity of the structure. An 18 percent reduction of the dead load was attained at the center of each span.

CONSTRUCTION

A major feature in the design of the bridges was to keep the structures as simple as possible to facilitate their construction. The cost of formwork was reduced because it was flat, without recesses for beams. Placement of reinforcement was simplified since no bars were bent.

The concrete was cast in place and the forms supported on a wood falsework. The cardboard tubes were placed between the positive and the negative reinforcement, and there were no problems with flotation during the concreting.

Cost

The average cost of the superstructure for these bridges, including the railings and the columns, was $7.82 per sq ft, of which $7.03 per sq ft corresponds solely to the slab.

CONCLUSIONS

The use of a reinforced concrete slab for the bridges of Via Monumental was the most convenient for the following reasons:

1. Reducing the total height of the bridges also reduced the height of the approach fills, which made more area available for urban development in the vicinity of the crossings.
2. The vertical curve of the grades in the bridges allowed the length of the approaches to be decreased.
3. The thin slabs provided the smallest possible obstruction to vision for the drivers of vehicles.
4. The inclined columns favored the behavior of the structure and also introduced a different type of aesthetic characteristic in the bridges, which was one of the effects sought by the investors.
5. The bridges were extremely economical, especially if the reductions in the cost of the approach fills is taken into consideration.

PROJECT CREDITS

The bridges were designed by Sáenz-Cancio-Martín-Gutiérrez, Ingenieros-Arquitectos, Havana, for Compañía de Fomento del Túnel de la Habana and Financiera Nacional de Cuba. The supervision of construction was handled by Armando Pérez Cobo and Manuel Vilaret, civil engineers, and Ignacio Ortega was the resident engineer. The bridges were built by Asociacion de Contratistas Independientes, S.A.

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Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1960, for publication in March 1961 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Montrose Avenue Bridge

M. SCHUPACK, *Journal*, Prestressed Concrete Institute, V. 5, No. 1, Mar. 1960, pp. 52-58

Describes design and construction of a three span cast-in-place prestressed concrete bridge with 154-ft spans.

New materials used in slab bridge (in Dutch)

A. H. VAN RIJS and A. LEEN, *Cement* (Amsterdam), V. 11, No. 4, Aug. 1959, pp. 369-375
Reviewed by JOHN W. T. VAN ERP

This prestressed, skew, hollow slab bridge had three continuous spans. New materials used in its reconstruction were: (1) large diameter (16 in.) cardboard tubes to form hollow slab; they were extensively tested for shape retention, waterproofness, and resistance to damage from a heavily vibrated low slump concrete; (2) 100-ton Freyssinet cables with special shallow cast steel anchor sleeves; (3) rubber-steel bearing plates consisting of three steel plates with rubber layers in between, the entire unit encased in rubber for corrosion protection.

Setting a span record: Prestressed bridge has suspended girders

Engineering News-Record, V. 164, No. 13, Mar. 31, 1960, pp. 30-34

Record-setting prestressed girders are only one interesting feature of the bridge across Lake Oneida at Brewerton, N.Y. In addition to containing the longest prestressed spans in the western hemisphere, 320 ft, none of its cantilever girders is supported directly from beneath, all are hung from diagonal tendons and rods; and its suspended span girders are set between the cantilever girders rather than on them, held in place by transverse prestressing tendons. The unusual design was dictated by erection problems.

Much of the article is devoted to precasting and prestressing of the members.

Trends in the development for construction and erection of concrete bridges (Entwicklungstendenzen im Bau und Montage von Massivbrucken)

HERMANN BAY, *Beton, Herstellung, und Verwendung* (Düsseldorf), V. 9, No. 5, May 1959, pp. 165-175
Reviewed by FERDINAND S. ROSTASY

Unlike steel bridges, the design and construction of centering are important items in the cost of a concrete bridge. The function of the classical centering for arch bridges, which served as an example for several decades, is described. Falseworks of famous European bridges are shown. After World War II the tubular steel scaffold became a serious competitor to wood centering. Its quick assemblage, adaptability, and reuseability are the major advantages over wood formwork. The neccessity to span inaccessible precipices and gorges led to the self-carrying scaffolds which are most-

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 32, No. 1, July 1960, *Proceedings* V. 57. Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are *not* available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

ly structural truss members themselves. Special methods of erection and assemblage are described by means of examples.

Colesberg Bridge

The Engineer (London), V. 208, No. 5398, July 10, 1959, p. 16
Reviewed by ARON L. MIRSKY

New two-lane, high-level highway bridge over the Orange River in the Union of South Africa, replacing old single-lane bridge which took 12 years to build at the end of the nineteenth century, is approximately 1300 ft long and consists of 14 spans (nine of 75.5 ft, two of 100.5 ft, and three of 125.5 ft) of reinforced concrete. Of the 13 piers, nine are of hollow reinforced concrete construction; the other four are larger, carry a 52 ft over-all double cantilever, and are prestressed by cables grouted 15 ft into the underlying bedrock to counteract any tension due to unbalanced loading of the superstructure.

Nordwestbogen Bridge in Berlin (prestressed-concrete rigid frame) [Das Bruckenbauwerk Nordwestbogen in Berlin (Spannbetonrahem)]

HANNS HEUSEL, *Der Bauingenieur* (Berlin), V. 34, No. 5, May 1959, pp. 169-178
Reviewed by ARON L. MIRSKY

Bridge over the Spree is part of a loop road around Berlin, which in turn is part of a projected high-speed highway net serving the city. It consists of 12 spans, in six sets of two spans continuous over and acting integrally with the intermediate pier. Both longitudinal and transverse prestressing was used. Since existing channel is a busy waterway, and temporary in-stream bents could not be used, construction of this portion featured a modified "free cantilever" approach: initial sections were cantilevered both ways from the central pier, while most of remainder was cable-supported in balanced fashion from temporary towers erected on that pier in the manner of the Severin Bridge across the Rhine at Cologne (*Engineering News-Record*, V. 164, No. 5, Feb. 4, 1960, pp. 34-36, 38); only the last sections were supported on temporary bents erected on shore. An interesting and ingenious solution to a challenging problem.*

*Details of the other designs submitted in the 1957 design competition will be found in *Beton-und Stahlbetonbau*, V. 53, No. 5, May 1958, pp. 106-117; Current Reviews, ACI JOURNAL, V. 30, No. 8, Feb. 1959 (*Proceedings* V. 55), p. 910.

Construction

Load-carrying structure of the new Hessian National Bank in Darmstadt (Die Tragkonstruktion des Neubaues der Hessischen Landesbank in Darmstadt)

A. MEHMEL and E. KERN, *Der Bauingenieur* (Berlin), V. 34, No. 5, May 1959, pp. 179-185
Reviewed by ARON L. MIRSKY

Structure contains two basements, a ground floor, four upper stories, and an attic. Plans called for a 15 x 30 m portion of the ground floor to be unobstructed; this was accomplished by suspending the four stories above the ground floor from the roof trusses, which are of reinforced concrete with prestressed concrete tension ties; the suspenders are also of prestressed concrete. The hip girder was also prestressed. Design and construction of the shear walls, the gable walls, and the stairway are also discussed.

L-shed at Durban harbour

Public Works of South Africa (Johannesburg), Nov. 1959, pp. 21-23
Reviewed by D. G. NORMAN

New ocean terminal at Durban, costing approximately $6,000,000, is now under construction. Design features variable depth rectangular prestressed girders carrying flat roof of precast slabs which forms car park. Other features are a helicopter platform and precooling and fruit handling facilities.

Telescopic caisson lighthouse

The Engineer (London), V. 208, No. 5407, Sept. 11, 1959, pp. 238-240
Reviewed by ARON L. MIRSKY

To reduce amount of hazardous on-site construction, Swedish lighthouse construction has now been developed

to an interesting point: the double telscopic caisson, consisting of two closed-bottom sections, one nesting inside the other. These are constructed on land, launched, and towed to the site; the outer caisson is then sunk onto a prepared bed, and the inner caisson (the tower) is then raised by pumping water into the space between the caissons, and concreted in place. Informative article describes lighthouses constructed from 1953 to 1959, stages in the development of this concept, and the construction methods used.

Regulating works of the Baix-le Logis Neuf canal, by La Compagnie Nationale du Rhone (Les travaux d'amenagement de la chute de Baix-le Logis Neuf, par la Compagnie Nationale du Rhone)

G. Gres, *Le Genie Civil* (Paris), V. 136, No. 9, May 1, 1959, pp. 193-200.
Reviewed by Aron L. Mirsky

Covers construction of the power plant-outlet valve-spillway portion of the project, and the tailrace. Considerable attention (especially photographic) is devoted to the formwork for the power plant.

Watertight deck slab

Contractors and Engineers, V. 56, No. 11, Nov. 1959, pp. 86-88

Describes the construction procedure for making a waterproof concrete deck strong enough to park automobiles. The roof slab is supported by T-girders, both of which were given a combination of post-tensioning and pretensioning. The columns, precast at the job site, were heavily reinforced.

Silo for sugar storage (inDutch)

M. Wierda, *Cement* (Amsterdam), V. 11, No. 4, Aug. 1959, pp. 334-336
Reviewed by John W. T. Van Erp

First silo for bulk beet sugar storage, built in the Netherlands by slip-form method, has 10,000 ton capacity. It has a 120 ft height, a 75 ft diameter, and was prestressed for crackfree walls. There is a centrally located vertical inspection shaft with an adjacent elevator and conveyor tower that was also built by the slip-form method. In view of the expected settlement, both silo and conveyor tower were built on independent foundations (concrete piles). The silo had no exposed steel due to the aggressive nature of sugar.

Corrugated arch shell construction

International Civil Engineer & Contractor (London), V. 11, No. 4, July-Aug. 1959, p. 58-59

Brief description of two corrugated concrete shell warehouses built in Paisley, Scotland, of 30,000 and 45,000 sq ft. The roofs are 2½ in. thick reinforced concrete shells spanning 100 ft. The Ctesiphon system conforming to an inverted catenary was used. The concrete shell was cast on a formwork of expanded metal which was draped to the shape of the corrugations, three to every 25 ft of width.

Lining of an aqueduct tunnel with precast reinforced concrete pipe (Stollenausbau durch Verwendung vorgefertigter Stahlbetonrohre)

H. Burkert, *Bauplanung-Bautechnik* (Berlin), V. 13, No. 7, July 1959, pp. 302-307
Reviewed by J. F. Leppmann

High strength precast concrete pipe, backfilled with lean concrete, were substituted for an initially planned cast-in-place concrete lining with a pneumatically applied inner mortar coating. The tunnel, nearly 2 miles long, had been completed before lining operations began, and stood up without timbering. The 8 ft-1 in. long pipe sections have a 6 ft-6 in. inner diameter and 5.5 in. wall thickness and were placed on prepared concrete saddles from a specially designed carriage which ran on a 24-in. gage track. The same track was used by a concrete mixer train from which the backfill concrete was placed pneumatically.

About half of the article describes the water proofing of the pipe, especially the pipe joints, against a water pressure of 43 psi. The abutting pipe end surfaces were provided with pre-

molded bituminous seal rings of initially rectangular cross section. After each pipe section was placed it was pressed against its predecessor with a longitudinal force of from 22,000 to 24,000 lb, squeezing the seal rings until they filled the joint completely.

Job-built jacks handle prefabricated roof forms

Conctractors and Engineers, V. 57, No. 5, May 1960, pp. 142-145

A sturdy, simple form system, prefabricated at floor level on the job and raised into place by job-built jacks, resulted in speedy and economical construction of a thin-shell hyperbolic paraboloical concrete roof for a 141 x 281ft warehouse. Each form section was 17 1/2 x 35 ft; eight of these panels were built.

Precast columns, tilt-up walls, and a cast-in-place floor slab completed the structure.

Construction Techniques

Foundations on cast-in-place piles

F. Derquı, Madrid, Spain, 12th Edition, 1958, 71 pp.

BRI Abstracts
Sept. 1959

The intends that these 130 articles forming the context for suggested international specifications will provide sound, experienced, and reliable information for designing pile foundations. This revised edition is the result of collaboration with engineers in several countries.

Pumped concrete (Pumpbeton)

E. Riesel, *Beton, Herstellung, und Verwendung* (Düsseldorf), V. 9, No. 2, Feb. 1959, pp. 52-58

Reviewed by Ferdinand S. Rostasy

Conveying of concrete by pumping is today, after initial failures, generally acknowledged. The pumps used are heavy-duty, piston type. The various pipes and connections permit contractors to adapt to any kind of project. Widely different opinions exist as to the efficiency of the pumps, but roughly 25 cu yd per hr are possible. The maximal length of transport is given as 650 yd, while for vertical transport 55 yd are mastered.

Uniformity is most important for pumping. Experience shows that plastic to medium stiff concrete is most suitable. Cements which give a workable mix should be chosen; a minimum cement content of about 4.5 sacks per cu yd of concrete is essential. The gradation of the aggregates has to guarantee a smooth flow without segregation. The author gives sieve curves with reference to German code DIN 1045. A suitable water-cement ratio in accordance with cement content and aggregate gradation has to be chosen. Plastifying and air-entraining agents can be applied successfully, accelerators cautiously.

Use of fluxed bitumen for curing concrete (in Russian)

V. Pankratov and B. Musin, *Avtom Dorogi*, V. 22, No. 7, 1959, p. 11

Road Abstracts
V. 27, No. 2, Feb. 1960

Specifications are given for the preparation of fluxed bitumen with 40 percent benzine A-66. The mix is sprayed on freshly laid concrete at a rate of 300 gal. per sq m. Greater strength of concrete is achieved by this method than by the use of moist sand or sawdust, with a 70 percent savings in the cost of curing.

"Pretub" method for the lining of circular galleries (Le procede "Pretub" pour la realisation du revetement des galeries circulaires)

J. Balensi, *Le Genie Civil* (Paris), V. 136, No. 10, May 15, 1959, pp. 224-225

Reviewed by Aron L. Mirsky

Describes method and apparatus for lining circular galleries in unstable ground with precast concrete voussoirs. Blocks are introduced at bottom of cage which consists of an inner and an outer ring, latter serving also as shield. Rings are equipped with rollers; blocks are moved toward crown by hydraulic jacks, all operations being performed

symmetrically about a vertical diameter, culminating in insertion of two half-blocks with a small hydraulic jack between them, activation of which ensures full seating of all blocks. Longitudinal movement of the rig is also accomplished by hydraulic jacking.

Internal vibration of concrete

J. KOLEK, *Civil Engineering and Public Works Review* (London), V. 54, No. 640, Nov. 1959, pp. 1286-1290

Describes tests of internal vibrators for use in the precast concrete industry. Object was to investigate possible methods of testing and determine relative efficiency of ten different vibrators. Most favorable method was based on ultrasonic pulse velocity in a block of concrete compacted by internal vibration.

Author concludes that of vibrators tested those having frequencies above 6000 rpm were most effective as was the case with those having heads 1.5 in. in diameter or greater. Ultrasonic pulse velocity measurements can distinguish quantitatively the effectiveness of a vibrator. Exact measurement of the parameters of vibration (amplitude, frequency, acceleration) was judged to be difficult and the results unreliable.

A-power plant takes shape on the Hudson

Engineering News-Record, V. 164, No. 11, Mar. 17, 1960, pp. 40-42

. . . Here's how they built the dome

WILLIAM HERSHLEDER, *Engineering News-Record*, V. 164, No. 11, Mar. 17, 1960, pp. 43-44, 46

First article describes general layout and construction of plant for Consolidated Edison Co., N. Y., at Indian Point. Second specifically describes erection of the precast concrete dome over the power reactor.

Reactor building is a circular cast-in-place structure 180 ft in diameter topped with a dome formed of precast ribs, acting first as simple beams springing from the top of the circular wall to a temporary platform on the tower of a central derrick then as fixed arches, covered with precast curved planking. The dome shape is neither a true sphere or a surface of revolution, providing interesting forming problems.

Site layout is described for fabricating and erecting both the spherical steel vapor container and the covering dome. Both were erected using the central tower-supported guy derrick erected at the center of the reactor structure.

Welding of reinforcing bars for concrete construction

J. F. RUDY, F. SUYAMA, and H. SCHWARTZBART, *Welding Journal*, V. 38, No. 8, Aug. 1959, pp. 335s-344s

APPLIED MECHANICS REVIEWS Mar. 1960

An investigation of the welding of reinforcing bars has been conducted on random lots, obtained from 11 different suppliers, and representing hard, intermediate, and structural grades, in three different bar sizes, embracing a range of chemical composition.

Several geometries of both lap and butt welds were investigated, totaling approximately 550 weldments.

A welding procedure is presented with which strengths up to 75,000 to 80,000 psi have been obtained in butt welds on bars of all analyses investigated, except structural grade, where 90 percent joint efficiency was obtained. These values represent joint efficiencies approaching 100 percent for intermediate and structural grade bars, and 60 to 80 percent for the hard grade bars, based on the ultimate strength as 100 percent efficient. If the yield point, which is the figure used for design calculations, is adopted as the basis for joint efficiency, these values run from about 105 percent for high grade steels up to about 175 percent for the intermediate and structural grades.

The procedures involved utilize either E7016 electrodes (70,000 psi ultimate, low hydrogen) or its iron-powder modification. The joint geometries used are conventional, with no preheat requirement. Limited evaluation of the procedure on bars which had been cooled to 0 F indicated no deleterious effects of low temperature.

Dams

Experimental method of investigating stresses in buttress dams

J. R. Rydzewski, *British Journal of Applied Physics* (London), V. 10, No. 10, Oct. 1959, pp. 465-469

Reviewed by Aron L. Mirsky

Gives details of method developed for investigating the stress distribution in a buttress dam web of constant thickness, built on an elastic foundation with properties different from those of the dam proper. Method utilizes Araldite models cemented to foundation blocks of another material (not necessarily photo-elastic), centrifugal loading, frozen-stress techniques, and a modified form of a readily available comparator such as used in machine shops for checking surface finishes, for obtaining the sum of the principal stresses at any given point directly. Developmental tests of model manufacture, loading, heating and cooling, etc., are described. Method promises to be useful, to yield results of high accuracy, and to be applicable to gravity dams.

Features of Lednock Dam, including the use of fly ash

Arthur C. Allen, *Proceedings*, Institution of Civil Engineers (London), V. 13 (Session 1958-59), June 1959, pp. 179-196

Reviewed by Aron L. Mirsky

Lednock Dam, a part of the North of Scotland Hydro-Electric Board's Breadalbane Scheme,* is a mass concrete buttress dam consisting of 13 diamond-head buttresses with solid gravity end sections, featuring extensive pressure grouting, horizontal construction joints, a single water seal of bitumen between adjacent buttresses and the gravity blocks, and an inspection shaft in the head of each buttress. Fly ash was used as replacement for 20 percent of the cement by weight, resulting in increased workability, slower hardening, reduced heat of hydration, and reduced cost. However, increased carbon content of the fly ash over that anticipated resulted in lower strengths; required minimum 28-day strength was thus set at 71 percent of all-portland cement concrete. Author notes that specific surface of fly ash is not a good criterion and suggests some other method of expressing the physical characteristics of this material be developed.

*cf *The Engineer* (London), V. 206, No. 5346-5349, 1958; Current Reviews, ACI Journal, V. 31, No. 5, Nov. 1959 (*Proceedings*, V. 56), p. 426

Why don't U.S. dambuilders build buttress type dams?

G. S. Sarkaria, *Engineering News-Record*, V. 164, No. 13, Mar. 31, 1960, pp. 30-34

Presenting the case for buttress dams, two types are discussed: massive-head and hollow gravity dams. Several deterrents to the use of buttress dams are discussed and the point made that they can be overcome by suitable design and modern construction methods. Discussion of design elements and costs is offered along with a list of the 30 buttress dams built or started in the western world since the early 1940's.

Cavitation damage of roughened concrete surfaces

Donald Colgate, *Proceedings*, ASCE, V. 85, HY 11, Nov. 1959, pp. 1-10

Author's Summary

Discusses an exploratory laboratory study concerning the evaluation of the cavitation potential of various types of roughened concrete surfaces. Methods of evaluating the data determined by the laboratory study, and means of presenting the results for use by design and field engineers, are discussed.

Simplified procedure for stress analysis of gravity dams

R. S. Sandhu, *Proceedings*, ASCE, V. 85, PO 6, Dec. 1959, pp. 173-187

Author's Summary

By applying the formula and tables derived in this paper a quicker solution of the "principal stresses," by the gravity analysis method, can be calculated for a gravity masonry dam. These short cuts although saving considerable man-hours do not impair the accuracy of the results.

Design

Folded plate design (in Dutch)

H. Grabowski, *Cement* (Amsterdam), V. 11, No. 5, Oct. 1959, pp. 460-465
Reviewed by John W. T. Van Erp

Method of analysis is developed by taking the normal stresses along the ribs and the transverse moments as unknowns. These unknowns are then determined by means of successive approximations. Article shows transverse moments that are due to displacement of ribs can be neglected. Method given can also be applied to cylindrical shells.

Design and selection of hyperbolic cooling towers

R. F. Rish and T. F. Steel, *Proceedings*, ASCE, V. 85, PO 5, Oct. 1959, pp. 89-117
Authors' Summary

The structural design of hyperbolic cooling towers is discussed with particular reference to wind stresses in the shell. Formulas are given to enable the size of cooling towers to be determined for a given cooling duty, and a method outlined for the selection of the most economic duty.

Ultimate load theory for concrete frame analysis

A. L. L. Baker, *Proceedings*, ASCE, V. 85, ST 9, Nov. 1959, pp. 1-29
Author's Summary

This paper deals with (a) an idealized conception of elastic members joined by plastic hinges; (b) basic parameters, minimum values, and theoretical assumptions for strength and deformation calculations of members and hinges; (c) the behavior of ultimate designs under working load.

Wind pressures in various areas of the U.S.

Guttorm N. Brekke, *Building Materials and Structures Report* No. 152, National Bureau of Standards, Apr. 1959, 8 pp., $0.15
BRI Abstracts June 1959

A procedure is described for developing a wind-pressure map that shows minimum resultant wind pressures (30 ft above ground) for design purposes. It is based on records of annual fastest-single-mile wind speeds at 155 U. S. Weather Bureau stations. In computing resultant wind pressures from these data, allowance was made for gusts and for building shape. A table of design wind pressures at various heights from less than 30 to more than 1200 ft above ground is provided for use with the map. Principal sources of strong winds affecting building constructions are reviewed briefly.

Design of folded plates

Eliahu Traum, *Proceedings*, ASCE, V. 85, ST 8, Oct. 1959, pp. 103-123
Author's Summary

Various design theories of folded plate structures are critically reviewed. The advantages and generality of the "method of particular loadings" are stressed and an outline of its application is given. The method is illustrated by a numerical example.

Properties of rectangular reinforced concrete beams under flexure (in Spanish)

Henrique Arnal Arroyo and Oscar Guerra Munoz, *Revista del Colegio de Ingenieros de Venezuela* (Caracas), No. 272, Nov. 1958, pp. 35-39.
Reviewed by Joseph J. Waddell

A group of tables for quick solution of the properties of reinforced concrete beams, such as effective depth, shearing stress, and reinforcing requirements.

Flat plates (in Spanish)

V. Martin Elvira, *Revista del Colegio de Ingenieros de Venezuela* (Caracas), No. 282-283, Sept.-Oct. 1959
Reviewed by Joseph J. Waddell

Known and exact solutions for elastic flat plates consists of complicated transcendental equations which are of limited value in many cases. This paper presents an exact algebraic solution, of the general case, for a plate subject to a load at right angles to its middle plane. The case of the rectangular flat plate subject to uniformly distributed load is presented, considering also the related parts as they may be

most frequently used in actual construction.

A comparison is made with the method of S. J. Fuchs in "Plates With Boundary Conditions of Elastic Support," *Separate* No. 199, ASCE, 1954, and with certain experimental data.

The author concludes that this method is more simple than the usual methods, but results are exact, being in conformance with carefully conducted tests. He suggests that further experimental work be done.

The three-span articulated beam

REGINALD G. ROBERTSON, *Proceedings*, Institution of Civil Engineers (London), V. 13 (Session 1958-59), Aug. 1959, pp. 467-476
Reviewed by ARON L. MIRSKY

For a three-span beam with a portion of the center span suspended from two cantilever arms, author develops expressions for the optimal span proportions, depths, and haunch sizes (required in the longer spans). These are then presented in the form of a design chart. A worked example is given, for a prestressed concrete beam; author indicates method is also applicable to ordinary reinforced concrete structures. Finally, a comparison is made with a continuous beam of uniform section and with a structure of three simple spans, both as previously designed by the author [*Proceedings*, ICE, V. 10, May 1958, pp. 19-38 and 39-58; Current Reviews, ACI JOURNAL, V. 31, No. 1, July 1959 (*Proceedings*, V. 56), pp. 89, 90].

Statistical approach to structural design

HSUAN-LOH SU, *Proceedings*, Institution of Civil Engineers (London), V. 13 (Session 1958-59), July 1959, pp. 353-362
Reviewed by ARON L. MIRSKY

Author proposes a rational design method, based on the principles of probability and statistics. He points out that "serviceability" would be more appropriate than "safety" as the criterion, and hence method is applicable to problems of buckling, minimum-weight design, fatigue, etc., and not only to the more common elastic or plastic design. Three examples are given: a steel beam of constant section, fixed at both ends; an axially-loaded reinforced concrete column; and the bearing capacity of a soil. Designs based on proposed method are shown to be more economical than usual solutions.

The discussion (March 1960, pp. 296-301) points up many of the feelings—pro and con—actuating engineering thought on the statistical approach to safety in structural design. Author's feelings are well summed up in his final phrase, ". . . . . the philosophy of the collapse-happy designers."

Bending moments on shell boundaries

H. H. BLEICH and M. G. SALVADORI, *Proceedings*, ASCE, V. 85, ST 8, Oct. 1959, pp. 91-101
AUTHOR'S SUMMARY

The structural analysis of thin shells is usually performed by determining the state of membrane stress due to the loads and by introducing bending corrections in the neighborhood of the shell boundaries. The corrections are required to satisfy the boundary conditions which cannot be satisfied by membrane stresses alone. The penetration of the bending stresses into the shell depends on the thickness and the dimensions of the shell, and also on the characteristics of the middle surface of the shell.

A comparison is made of bending stresses due to an applied boundary moment in flat plates, cylindrical shells, rotational shells, and saddle shells. The comparison illustrates the mechanism of bending stress penetration in the most commonly used types of thin shells.

The behavior of the shallow hyperbolic paraboloid supported along four straight generatrices is studied by means of Vlasov-type equations. The boundary disturbances to be added to the membrane stresses are known quantitatively in a rather special case, and qualitatively only in the general case. Their determination remains an open problem.

Relaxation methods for two-dimensional plate problems

J. A. N. Lee, *Engineering* (London), V. 187, No. 4865, June 5, 1959, p. 752*
Reviewed by Aron L. Mirsky

Demonstrates that relaxation methods give values of deflections at nodal points of flat plates under concentrated loads normal to the surface, which compare favorably with experimental results, provided the deflections are small compared to the plate thickness, the grid size is small, and the residual is well dispersed. Salvadori's (1951) extrapolations are shown to lie quite close to results obtained by relaxation methods.

*Digest of paper obtainable from The University, Nottingham, England (10 s).

Designing flat plates and slabs

E. Vernon Konkel, *Consulting Engineer*, V. 13, No. 5, Nov. 1959, pp. 110-113
Reviewed by Aron L. Mirsky

While use of flat plates and flat slabs results in maximum economy, both initial and long-term, due to the shallowness of the structural system, that very shallowness can lead to trouble due to excessive deflections. Article summarizes the various causes of excessive deflection and suggests methods of preventing or minimizing such deflection. Both the novice and the expert designer should benefit from reading this paper, to say nothing of the owners of such structures.

Initial and subsequent deflections of reinforced concrete beams in state II; Proposals for limitations and simplified proof (Anfangliche und nachtragliche Durchbiegungen von Stahlbetonbalken in Zustand II; Vorschage fur Bregenzungen und vereinfachte Nachweise)

F. Leonhardt, *Beton und Stahlbetonbau* (Berlin), V. 54, No. 10, Oct. 1959, pp. 240-247
Reviewed by Rudolph Szilard

The determination of deflections, neglecting the cracked tensile zone, gives different results than that required by the building codes. Propositions for alteration of the present German Building Code (DIN) requirements have been worked out by the author, including a simplified method which can be utilized for checking deflections caused by initial loading, long-time loading, creep, and shrinkage.

Method for establishing the dimensions of railway slab bridges with stiffened edges (in Hungarian)

O. Halasz, *Proceedings*, Technical University for Architecture, Civil and Transportation Engineering, V. 4, No. 2, 1958, pp. 3-12
Hungarian Technical Abstracts V. 11, No. 4, 1959

A method is presented for determining the dimensions of reinforced concrete railway slab bridges with stiffened edges. Proceeding from the theory of thin slabs, the stresses arising in a slab freely supported by the abutments and elastically stiffened by the border ribs are analyzed by the use of infinite series. Tables and diagrams are furnished on the maximum bending moments developing in the middle of the slab and in the border ribs for various conditions of stiffness and side ratio. The tables and diagrams also cover size ratios which have not been published in literature. The depth of railway slab bridges can be reduced by the proposed method.

Elastic deformations and forces in a circular, encastre, symmetrical arch loaded by a constant normal force N and a lateral uniformly distributed load p; stability of elastic equilibrium (Deformations elastiques et efforts dans un arc circulaire, encastre, symetrique, soumis a un effort normal N constant et a une charge laterale p repartie uniformement; stabilite de l'equilibre elastique)

P. Layrangues, *Annales des Ponts et Chaussées* (Paris), V. 129, No. 3, May-June 1959, pp. 323-344
Reviewed by Aron L. Mirsky

A constant normal force acting parallel to the neutral surface is shown to have a considerable effect on the lateral stresses and strains in a circular arch of large span carrying a uniform load

acting perpendicular to the plane of the neutral fiber (i.e., normal to the plane of the arch). Formulas are derived which are applicable to the investigation of the elastic stability of such a system, and to the calculation of the critical normal stress in terms of the arch geometry and the properties of the section. A numerical example demonstrates striking similarity of results to those obtained by Timoshenko-Hencky methods. While derivation is based on arches of constant section, a general theory to include variable flexural and torsional properties and variable normal force is possible.

Direct analysis of doubly-curved shells

S. P. Banerjee, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Dec. 1959, pp. 417-418
Author's Summary

The simultaneous solution of the differential equations of equilibrium of a shell element is rather difficult and it is therefore usual to have recourse to some iterative method to obtain the results indirectly. The paper presents an alternative method which gives a direct solution.

Loading capacity of the compression plate of reinforced concrete T-beams in simple bending (Die Trangfahigkeit der Druckplatte von T-Balken aus Stahlbeton bei einfacher Biegung)

Gottfried Brendel, *Bauplanung-Bautechnik* (Berlin), V. 13, No. 10, Oct. 1959, pp. 458-464
Reviewed by J. F. Leppmann

The design of T-beams is usually reduced to that of rectangular beams of the same depth by introduction of the so-called "effective width." In view of wide differences between the design specifications of various countries on the subject, the author presents a group of diagrams and tables which permit a numerical evaluation of the effective width as a function of the geometrical proportions and the type of loading for both isolated and multiple T-beams.

The calculation proceeds in two steps: the first is based on the known solution of the problem for infinitely flexible plates, the second considers their finite stiffness. Elastic-linear behavior is assumed in both. The results are compared with design specifications of numerous countries and are supported by tests made by the author.

Unfortunately, the formulas the author has used in the first step are not given nor are the numerical values used in the second step sufficiently explained. If one wishes to check the author's premises and the applicability of his methods to the solution of specific problems he has to resort to the sources listed in the bibliography, not all of which are readily available in English speaking countries.

Elements of reinforced concrete

Sylvan P. Stern, Prentice-Hall, Inc., New York, 1959, 444 pp., $10.60

A good text for a first course in reinforced concrete—provided that it is desired to apply only the working stress method of design; there is no mention of ultimate strength design. The author carefully takes the student through design of flexural members, utilizing primarily the method of transformed section. Beams with axial load are not considered.

Bond, shear, diagonal tension, and anchorage are well covered. The chapter on columns deals only with axially loaded columns—nothing on eccentrically loaded columns, so the "cracked section method" is not covered.

The chapter on retaining walls is based on the section on retaining walls and abutments in the American Railway Engineering Association manual. Other sections are all in accordance with the provisions of the ACI Building Code, ACI 318-56. Examples are carefully worked out in detail with all information given. One chapter takes the student through the foundation design for a steel-framed building. The chapter on detailing and drafting of reinforced concrete structures fol-

lows the *Manual of Standard Practice for Detailing Reinforced Concrete Structures* (ACI 315-57).

Apparently this book was designed for use in an elementary or introductory course to reinforced concrete design and should serve its purpose well. After developing the theories, the author demonstrates the practical application in the foundation design and then presents a working set of drawings.

Materials

Evolution of the cement industry and its adaptation to underdeveloped countries (L'evolution de l'industrie cimentiere et son adaptation aux pays sous-developpes)

ALFRED BAOUMAN, *Le Génie Civil* (Paris), V. 136, No. 11, June 1, 1959, pp. 249-250
Reviewed by ARON L. MIRSKY

Discusses requirements of modern cement plants and shows that such plants are suitable for establishment in underdeveloped sections of the world (Africa, Asia) possessing the requisite raw materials.

Lower cost aggregate production through automation

J. E. DOYEN, *Pit and Quarry*, V. 52, No. 9, Mar. 1960, pp. 112-117

A review of various applications of automation to quarrying and aggregate production operation. Four systems described are: (1) primary system, (2) secondary and tertiary crushing system, (3) bin storage system and, (4) blending and loading system.

Basic rotary kiln design for lightweight aggregate

M. F. PARSONS, *Pit and Quarry*, V. 52, No. 9, Mar. 1960, pp. 100-104

Describes a setup for expanding clay and shale in rotary kilns to produce high-strength lightweight aggregate. Some basic characteristics of both materials are noted. Five required features of an efficient kiln are that it must be (1) equipped with a cooler, (2) small in relation to its production, (3) operated at high inclination and high speed, (4) equipped with internal preheaters and dams to build high loads, and (5) equipped with a mechanical draft.

Note

Copies of books and articles reviewed are not available from ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In some cases ACI can furnish addresses of publications added later.

Cement hydration (Die Hydratation des Zements)

Zement und Beton (Vienna), No. 16, July 1959, pp. 3-37
Reviewed by FRITZ KRAMRISCH

The entire issue is devoted to the publication of lectures, reports, and discussions regarding the hydration of cement, presented at a conference held for this sole purpose in Vienna.

The articles deal with the chemical and physical characteristics of the hydration products of cement, findings regarding the hydration of calcium silicates, nonselective hydration of cement minerals, reactivity of gelwater, and the adsorbtion of the hardened cement.

To investigate the strength of the cement paste, tests on small test cylinders (11.3 mm diameter and length and 1 sq cm cross sectional area) are described and their advantages and shortcomings discussed.

Another article deals with the examination of hydrated cements under the electron microscope. It describes the preparation of the samples and the test arrangement and procedure. This article is accompanied by a number of interesting photographs taken with the electron microscope showing highly

magnified shapes, textures, grids, and mutual arrangements of the finest cement grains.

Use of fly ash in brick and block for construction (L'utilization des cendres de thermocentrales pour la fabrication des briques et des blocks de construction)

MANOLESCU, GEOGESCU, and GIVULESCU, *Revue des Materiaux* (Paris), No. 520, Jan. 1959, pp. 10-15; No. 525, June 1959, pp. 147-150
Reviewed by PHILLIP L. MELVILLE

Tests were performed on the use of fly ash from lignite. Best results were obtained with the following mix (by weight): 8 parts ash, 1 part dry, fresh unslaked lime, ½ part plaster, and 2½ parts water. Optimum curing was with steam at 80 C for 16 hr and 48 hr under water. Resulting block had a compressive strength of 140 to 160 kg per sq cm, good durability, insulating properties, and low cost.

An industry first: HMS for crushed stone

KNEELAND A. GODFREY, JR., *Rock Products*, V. 62, No. 11, pp. 96-99, 102

The crushed stone industry has finally come to using heavy-media separation. This Illinois plant installed a new plant to upgrade (at 50 tph) chert-bearing limestone. The plant not only removed all but 0.2 percent of the 3 to 4 percent chert, but it also removed some lightweight gray limestone of a generally higher specific gravity.

Use of fly ash in concrete by Ontario Hydro

J. N. MUSTARD and C. MACINNIS, *The Engineering Journal* (Montreal), V. 42, No. 12, Dec. 1959, pp. 74-79
Reviewed by ARON L. MIRSKY

Detailed resumé of Ontario Hydro's experience with fly ash in grouts and concrete, on the following projects (in chronological order): Niagara (Sir Adam Beck No. 2), St. Lawrence (Robert H. Saunders), Whitedog and Caribou, and Otter Rapids. Advantages cited are reduced temperature rise, increased workability, somewhat increased setting time, increased sulfate resistance, economy; disadvantages are slower strength development and increased form pressures due to lengthened setting time. Also described is batching of the ash as a slurry, developed on the St. Lawrence project and used successfully on succeeding projects.

A check list for concrete admixtures

A. G. LESSEL, *Concrete*, V. 67, No. 9, Sept. 1959, pp. 18-19

The problem of determining the relative merits and qualifications of concrete admixtures can be costly and confusing for those who haven't kept fully informed. To help facilitate this problem, a comprehensive check list is provided for the prospective user. It lists important properties and qualities that should be examined prior to accepting an "alloy" so as to help eliminate undesirable or harmful effects.

Study of the behavior of colored cements (Etude du comportement des ciments colores)

M. VENAUT, *Revue des Materiaux* (Paris), No. 524, May 1959, pp. 117-122
Reviewed by PHILLIP L. MELVILLE

Two yellow, one red, one black, and one green mineral pigments were added in variable quantities to white portland cement of three different finenesses. Tests were performed on shrinkage, swelling, and strength. The effects of the pigments was found to be minor.

Lime slag cement for use in masonry: IV

SHOICHIRO NAGAI and YOSHIMICHI IROKAWA, *Sekko to Sekkai* (Tokyo), No. 41, 1959, pp. 19-24
CERAMIC ABSTRACTS
Nov. 1959

Two series of cement samples were prepared (by weight): (1) 60 parts of granulated blast-furnace slag, 20 to 30 parts of fly ash, and 20 to 10 parts of $Ca(OH)_2$; (2) 50 to 70 parts of white

portland cement and 50 to 30 parts of pulverized slaking residue of lime. Effects of several different kneading solutions on the setting, hardening, and especially on the formation of efflorescence were tested. The use of H_2O containing 2 percent (weight) anhydrous Na_2SO_4 or the immersion of hardened 7 day old specimens in water is effective in preventing the formation of efflorescence.

Influence of airing of clinkers and cements on their physical and mechanical properties (Influence de l'eventment des clinkers et des ciments sur leurs proprietes physiques et mecaniques)

M. VENUAT, *Revue des Materiaux* (Paris), No. 523, Apr. 1959, pp. 79-88
Reviewed by PHILLIP L. MELVILLE

Changes in setting time, strength, and shrinkage cracking were related to airing of cement at 20 C, 65 percent relative humidity for 6 months. Clinker was found unaffected by storage if the relative humidity was less than 75. It was found that the finer the cement, the more deleterious was the mixing. The use of hydrophobic cement resulted in lower strengths.

Sawdust concrete

Constructional Review (Sydney), V. 33, No. 2, Feb. 1960, pp. 30-32

A brief summary of information on sawdust concrete covering strength, usage, drying shrinkage and moisture movement, pre-treatment of sawdust, proportions, mixing, placing, curing, and a sawdust-sand mix.

Modern materials: Concrete interlay; thermoplastic membrane

Contractors Record (London), V. 70, No. 7, 1959, pp. 22-23
ROAD ABSTRACTS
Aug. 1959

A brief description is given of a multi-ply thermoplastic material, "Lubrithene." It can be used between a concrete surfacing and the base to reduce friction while the concrete is setting. It is claimed that it permits a design strength of 2500 psi at 28 days, without the risk of early uncontrolled cracking and thus leads to a saving in the amount of cement needed. A reduction in the amount of steel reinforcement and in the number of joints is also said to be possible when this membrane is used.

Improvement of quality of concrete through admixtures (Gutesteigerung des Betons durch Zusatzmittel)

G. ROTHFUCHS and W. WEGENER, *Beton, Herstellung, und Verwendung* (Düsseldorf), V. 9, No. 3, Mar. 1959, pp. 73-79
Reviewed by FERDINAND S. ROSTASY

The authors describe admixtures with respect to their effects and use in concrete. They cover: plastifiers, air-entraining agents, air-entraining plastifiers (a combination of the first two, but with moderate air-entraining action), sealers (admixed to concrete), retarders, accelerators, and admixtures to grouts used to fill post-tensioning ducts. When special cements (low in C_3A) are used, admixtures should be applied cautiously.

Aggregate grading by method of least squares (in Swedish)

ERVIN POULSEN, *Nordisk Betong* (Stockholm), V. 4, No. 1, 1960, pp. 59-67
Reviewed by MARGARET CORBIN

Determination of the grading of combinations of different aggregates is treated analytically. The square of the deviates by weight from a given grading should be at minimum (the method of least squares). A formula is developed and numerical examples presented.

Tuff concretes (in Hungarian)

J. UJHELYI, *Magyar Épitöipar*, V. 7, No. 10-12, 1958, pp. 466-473
HUNGARIAN TECHNICAL ABSTRACTS
V. 11, No. 3, 1959

Research has been conducted on the production of tuff concretes, their properties, and possibilities of use. The applicability of the tuff aggregate and the attainable strength of the concrete were established. Comminution of the aggre-

gate, optimal grading, the necessity of previous saturation with water, and other technological problems of concrete preparation, e.g., heat curing, are dealt with in detail as well as the factors influencing the characteristics of concrete. Primarily the strength, weight per unit volume, moldability, and resistance to frost must be considered, other properties (such as heat insulation) being of secondary importance.

According to full-scale experiments a relatively high proportion of fine grains (minimum 30 percent of 0-1 mm size grains) is required but even this kind of aggregate must be previously saturated with water and mixed in a continuous mixer. Tuff concrete block may be economically produced both in small and in large plants. The weight per unit volume of the produced tuff concretes is 1400-1600 kg per cu m when using 200 kg per cu m of portland cement, the 28-day compressive strength being 100 kg per sq cm.

Magic memory solves raw mix problems

LEROY W. WEEKS, *Rock Products*, V. 63, No. 4, Apr. 1960, pp. 85-89

A digital computer and linear programming technique is being used by a cement company to solve raw mix problems. It is expected that use of the computer in running mix calculations will extend the useful life of the raw material deposits and help minimize quarrying costs.

Treating foamed concrete with CO_2 in the manufacture of insulating units (Le traitement du beton mousse par CO_2 pour la fabrication des elements d'Isolation thermique)

V. SLATANOFF and N. DJABAROFF, *Revue des Materiaux* (Paris), No. 522, Mar. 1959, pp. 72-75

Reviewed by PHILLIP L. MELVILLE

Six different foamed concrete mixes were prepared with variable amounts of lime, fly ash, and fine sand. Specimen were cured at 25 to 30 C and 60 to 70 percent relative humidity for 20 to 44 hr, then immersed in CO_2 for up to 30 hr. Results indicated that the resulting concrete was similar to autoclaved cured concrete.

Pavements

Concrete and soil-cement roads

W. P. ANDREWS, Contractor's Record Limited, London, 1960, 157 pp., 30s

This book gives a summary of new information on the design and construction of concrete roads and of concrete, lean concrete, and soil-cement bases for asphalt surfacings. It is based primarily on the work in England but also gives experience in these forms of construction in other countries. The book deals with everyday practical problems but also gives information on the theoretical side of road design. Construction details for both manual and mechanized processes are given. It includes details of special methods, such as prestressed concrete, and of roads for special purposes, such as concrete farm roads, and gives information about the maintenance and repair of concrete roads and concrete curbs.

Some results of recent tests in the field of concrete pavement construction (Einige Ergebnisse aus neueren Versuchen fur den Beton-Strassenbau)

G. WEIL, *Beton, Herstellung, und Verwendung* (Düsseldorf), V. 9, No. 1, Jan. 1959, pp. 3-12

Reviewed by FERDINAND S. ROSTASY

Relates experiences with the German autobahnen. A major cause of cracks and damage to pavement is frost action. Several examples illustrate possibilities which might lead to damage. Another reason is settlement of subsoil. Minimal pavement thicknesses and the actual loads as well as their frequency are discussed. Although standard of pavement contractors is high, current control of concrete quality is essential. Several nondestructive methods suitable for this purpose are described. A certain concrete strength has to be guaranteed, not only because of traffic action, but also to withstand thermal

stresses. Test results are presented which show variation of temperature across pavement thickness. "Black-topping" protects concrete from excessive thermal stresses. The influence upon formation of surface ice is discussed. Finally, the author deals with joints and the action of dowels. Sleepers underneath the joints must be sufficiently wide. Photoelastic tests show that pressure underneath the joint is uniformly distributed due to the sleepers.

Surface failures of concrete pavements and their repair (in Russian)

L. I. Goretskii, *Avtomatika Dorogi*, V. 22, No. 3, 1959, pp. 22-23

Road Abstracts Sept. 1959

The results are summarized of laboratory experiments carried out from 1956 to 1958 in which the effect of certain additives and treatments was investigated in regard to the resistance of concrete surface to scaling. Field research on the repair of surface damaged by scaling is discussed. For repairing damage 0.8 to 2.0 in. deep the application of mortars with various additives including latex, polyvinyl acetate, calcium chloride, and calcium nitrate is recommended. For superficial damage up to 0.8 in. the saturation of the surface with fluosilicate salts of magnesium or liquid siliceous compounds proved to be effective.

Construction of prestressed concrete road surfacing (in Russian)

I. I. Tetyutskii, V. N. Finashin, and V. G. Maidel, *Avtomatika Dorogi*, V. 22, No. 4, 1959, pp. 7-9

Road Abstracts Sept. 1959

An illustrated description is given of the design developed by Dormostproekt and used in the construction of a 560-ft experimental road section in Moscow. Slabs measuring 328 x 23 ft and 230 x 23 ft with a thickness of 6 in., reinforced with 153, 4- and 5-mm rods, were laid on bituminized paper over a 14-in. sand layer. A prestress of 214 tons was achieved by the use of eight angle braces each consisting of metal piles 18 ft and 10 ft in length driven into the foundation at an angle of 45 deg. The technique proved to be effective with increased economy in concrete and construction costs when 5 to 8 percent more than design stress was applied to allow for movement of the supports.

Post-tensioning for taxiway pavement

Contractors and Engineers, V. 56, No. 11, Nov. 1959, pp. 27-29

The U.S. Army Corps of Engineers have constructed at Biggs Air Force Base, El Paso, Tex., a 1550 ft long, 25 ft wide, 9 in. thick, prestressed concrete taxiway. The purpose in using a prestressed taxiway is to compare its durability, etc., to a 24-in. nonreinforced slab and to a 19-in. reinforced slab.

Construction procedures and practices are also given.

Two home-made machines to cut paving cost

Construction Equipment, V. 21, No. 2, Feb. 1960, pp. 59-60, 62

Specifications on this unreinforced concrete pavement, 11 in. thick in the main line lanes and 9 in. in the service roads, called for the insertion of asphalt paper transverse joints in the pavement every 15 ft. The contractor built a machine which automatically inserts the paper joints into the fresh concrete about as fast as the crew can load it. The other contractor-built machine handles heavy curb form sections.

Concrete surfacings under dynamic loads (in Russian)

A. P. Sinitsyn and G. I. Glushkov, *Avtomatika Dorogi*, V. 22, No. 4, 1959, pp. 25-27

Road Abstracts Sept. 1959

The results are recorded of research carried out in spring and autumn on experimental concrete road sections in different climatic regions of the USSR. The strength of the surfacing was de-

termined by the application of static loads at the slab corners. Single- and two-layer surfacing with different types of joint and reinforcement were tested to destruction by a test vehicle travelling at various speeds with a variable load of 30 to 60 tons. Equipment to measure load stress and flexure was built into the surfacing. A formula is presented for relating deformation to the number of passes. The most stable surfacing under repeated dynamic loading proved to be a two-course surfacing on a rigid foundation and a single-course surfacing on a sand layer with dowel joints and deformed bar reinforcement along the slab edge at the joint.

Montreal-Laurentides superhighway (L'autoroute Montreal-Laurentides)

OLIER MATHIEU, *L'Ingénieur* (Montreal), V. 45, No. 178, Summer (June) 1959, pp. 20-28
Reviewed by ARON L. MIRSKY

General description of Canada's first toll superhighway, a divided-lane, controlled-access road complete with cloverleafs and all the other amenities of modern high-speed throughways. The highway runs approximately 28 miles from the Boulevard Métropolitain in Montreal to St. Jérôme, where it ties into Highway 11 serving the Laurentian resorts. Article covers history, selection of route, right-of-way purchases, design and construction, and other phases of the project.

Development of Gatwick Airport

FREDERICK S. SNOW and NORMAN J. PAYNE, *Proceedings*, Institution of Civil Engineers (London), V. 14 (Session 1958-59), Sept. 1959, pp. 43-66
Reviewed by ARON L. MIRSKY

Gatwick Airport, some 20 miles south of and serving London, contains much of engineering interest. The structural features of the aircraft service depot have already been covered in the technical literature;* paper under review discusses other features, such as the reasons for the selection of unreinforced dowelled concrete pavement, the design and construction of the pavement joints, concrete mix proportions, the method of darkening the concrete, and concrete testing, as well as many features of primarily aeronautical interest.

*e.g., "Structure of Aircraft Service Depot at Gatwick for Transair Ltd.," W. J. M. Haines and A. J. Harris, *Proceedings*, ICE, V. 12, Feb. 1959, pp. 187-196; Current Reviews, ACI JOURNAL, V. 31, No. 6, Dec. 1959 (*Proceedings*, V. 56), p. 565.

Dual-drum paving costs vs. mixing time

MORGAN J. KILPATRICK, *Public Roads*, V. 31, No. 1, Apr. 1960, pp. 12-15
AUTHOR'S SUMMARY

Skilled paving contractors have consistently produced portland cement concrete in dual-drum pavers with controlled mixing time of 45 sec or less, including transfer time. The increased tempo required for operating the paver with a short cycle and the consequent short mixing time has been achieved in part through use of larger and faster batch trucks, and in part through the contractors' demonstrated awareness of the importance of eliminating unnecessary delays which consume valuable production time.

Monetary savings ranging from 10 cents to $3 per cu yd of concrete might be realized if mixing time specifications were reduced to 45 sec from their present levels of up to 120 sec. In the current highway program, millions of dollars might thereby be saved annually.

New runway at Marseilles-Marignane airport (in French)

P. MARIE and H. ROYER, *Travaux* (Paris), V. 43, No. 294, Apr. 1959, pp. 199-225
Reviewed by ETIENNE D. ROLIN

The concrete runways are designed for a single 30 ton wheel load at a tire pressure of 130 psi. Pavement sections are designed on the basis of CBR and Westergaard methods. Slabs are 10 in. thick, cast over ⅜ in. thick asphalt impregnated sand overlying the base course. Great importance is placed on the strength of the ⅜ in. layer of sand in order to level the surface of the base course so as to obtain a uni-

form slab thickness and thereby inhibit cracks. It also acts as a seal course.

Edges of slab are doweled with ⅝-in. smooth steel rods every 3 ft but without keys. Sawed contraction joints are ¼ the depth of the slab; this was more successful in forcing cracks to occur at the joints than the 1/6 depth previously used.

A mathematical study of the behavior of joints in concrete pavements is included which concludes that maximum adherence of slab to base course is advisable.

Precast Concrete

Block testing

W. C. Reid, *Concrete*, V. 67, No. 9, Sept. 1959, pp. 20-22, 40, 41

Plant compression testers enable concrete block manufacturers to check the quality of their product during important stages of manufacture. Information is given on how to use a block testing machine, such as preparation of block, operation, loading and preloading, rates of loading, and other phases of testing.

Rationalization of transport in the production of prefabricated elements and on the construction site (Transport-rationalisierung bei der Herstellung von Beton-Bauelementen und auf der Baustelle)

W. Teigelkamp, *Beton, Herstellung, und Verwenduna* (Düsseldorf), V. 9, No. 2, Feb. 1959, pp. 41-51

Reviewed by Ferdinand S. Rostasy

Deals with the present production of prefabricated elements of any kind in Germany. The author forecasts a radical change in organization of building activity if prefabrication is applied on a large scale. As an example the successful use of pavement elements is described.

The viewpoints for rational planning in a precast manufacturing plant as well as the economic flow and handling of the cement, aggregates, and finished products are discussed. Special attention is paid to transportation within the plant. Equipment successfully used is described. Loading and transportation of the elements to the construction site have to be in accordance with the placing progress. Placing equipment is described for several kinds of projects.

Prestressed slabs are slip-formed

Contractors and Engineers, V. 56, No. 11, Nov. 1959, pp. 44-46

This unique casting machine turns out hollow core prestressed precast floor and roof units by employing the slip-form technique.

A traveling gantry deposits three different layers of zero slump mixes as it moves along 328-ft casting beds. As the concrete is layed in position, core holes are formed by 6 ft long tubes that are pushed through the concrete by the same gantry. After curing, concrete saws cut the continuous slab to desired lengths.

Making spun concrete columns

International Civil Engineer & Contractor (London), V. 11, No. 3, May-June 1959, pp. 56-60

Reviewed by F. F. Gauld

An account of the manufacture of centrifugally cast concrete lighting standards in Great Britain. The article gives short descriptions of the various stages of manufacture and emphasizes the superiority of this method for producing high density concrete. Simulated traffic impact test showed the prestressed columns to be at least the equal of conventional reinforced concrete units.

Non-steel cylinder prestressed concrete pipes

S. R. Hubbard, *Proceedings*, ASCE, V. 85, PL 3, Oct. 1959, pp. 29-46

Noncylinder prestressed concrete pipe (pipe manufactured without the use of a steel cylinder membrane) will, if properly designed serve indefinitely. If concrete bells and spigots are provided, there will be little possibility of corrosion since there is no exposed metal.

Prestressing of the pipe must be done in such a way that safe values of con-

crete stresses are not exceeded. The procedures outlined here take time values and accompanying stress losses into account. Nomograms are included for computing stresses due to external loads.

Prestressed Concrete

Prison cell block built of prestressed concrete

Contractors and Engineers, V. 56, No. 11, Nov. 1959, pp. 82-84

The state prison at Carson City, Nev., has recently added a two-story maximum security cell block to its grounds.

The walls, roof, and floor were precast and prestressed 220 miles away and then hauled by truck to the work site. The advantages of this procedure were the rapid rate of erection and the minimum of construction personnel and equipment.

Optimum dimensions for prestressed concrete sections (Dimensionamiento optimo de secciones de hormigon pretensado)

JOSE MARIA URCELAY, *Bulletin* No. 193, Instituto Técnico de la Construccion y del Cemento (Madrid), 1958, 100 pp

Reviewed by ROGER DIAZ DE COSSIO

Based on a rather complicated stress-strain curve for the concrete in compression obtained by Torroja, and on the usual assumptions of linearity of strains and no tension in the concrete in flexure, the author obtains expressions and graphs for the most economical I-sections of prestressed concrete with and without unprestressed steel. The presentation is interesting because a factor of safety is explicitly considered. Only the conditions for pure flexure are obtained.

Prestressed steel beams (in Dutch)

A. DOBRUSZKES, *Cement* (Amsterdam), V. 11: No. 4, Aug. 1959, pp. 381-386; No. 5, Oct. 1959, pp. 449-459

Reviewed by JOHN W. T. VAN ERP

Standard wide flange steel beams are deflected under a vertical load and the tension flange is then encased in concrete. After taking off the load a prestressed beam results with considerably increased stiffness, about 1½ times greater than the original steel beam. These beams, named Preflex, employ high strength steels and are used primarily in conjunction with concrete columns. Beam design methods are given, including those for a continuous multistory frame. Examples of actual structures are shown such as buildings, bridges, and viaducts, demonstrating speed of erection and the reduced construction height.

1,850,000-gal. oil storage tank of prestressed concrete (in Norwegian)

E. KALHAUGE and ROLF OLSEN, *Betongen Idag* (Oslo), No. 3, 1958, pp. 69-74

Reviewed by ANTON HESSELBERG

The tank, built for a cement plant, is the first large oil storage tank of prestressed concrete, built according to the Freysinnet system, in Norway. In use since March, 1957, it has shown no signs of leakage. The foundation consists of ordinary reinforced concrete. The walls of the tank are prestressed horizontally with cables, vertically with bars. The top and bottom of the tank have ordinary reinforcing except for prestressing cables in the edge beams. The tank is isolated from the foundation with asphalt, asphalt paper, and a 2-in. Tresonite plate (woodwool).

Development of specifications for post-tensioned prestressed concrete (in Spanish)

LUIS A. PIETRI-LAVIE and FINN FREDRIKSON, *Revista del Colegio de Ingenieros de Venezuela* (Caracas), No. 270, Sept. 1958, pp. 4-7

Reviewed by JOSEPH J. WADDELL

These specifications embody the design and use of post-tensioned concrete for all types of structures. They are divided into six general sections: (1) contents; (2) definitions, including forces and losses; (3) materials, steel tendons (including wires, alloy steel bars, and strand) and anchorages, (joints in tendons, grouting, reinforcing steel, and concrete); (4) design stresses

in steel and concrete; (5) design details; and (6) construction.

Reference is made to the recommended practices of ACI, and the ACI-ASCE Committee 323 report to fill out details not spelled out in these specifications.

Properties of Concrete

Investigation into the relationship between the transverse strength and the compressive strength of cement mortar

M. F. KAPLAN, *Bulletin* No. 17, National Building Research Institute, Pretoria, Mar. 1959
Reviewed by D. G. NORMAN

Object was a new standard cement test. Correlation was good. Transverse strength is proportional to the 2/3 power of compressive strength and the proposed new test would record changes of over 150 psi of equivalent compressive strength.

Effect of the clinker minerals of portland cement on the properties of steam-treated concretes (in German)

SIEGFRIED REINSDORF, *Silikattechnik* (Berlin), V. 10, No. 5, 1959, pp. 260-265
CERAMIC ABSTRACTS Mar. 1960

Author investigated the composition and properties necessary in portland cements to obtain high strength under hydrothermal treatment. Alumina cements are particularly suitable for the steam treatment of concrete. The C_3A content of the clinker should be 10 to 15 percent with 5 to 7.5 percent SO_3. The proportion of C_2S in the basic silicate mix should be at least three times the C_2S content. Cements with a ratio of C_2S: C_3S of 6.0 gave favorable results. The lime ratio of the cements should be high for the best results. Free lime in the concrete was found to be harmless and the C_4AF content of the clinker did not affect the early strength of steam treated concretes. The cements should be ground fine, with a residue of 1.5 to 3.0 percent on 0.09 mesh (DIN 1171). The addition of fine quartz meal or blast-furnace slag to portland cements resulted in lower strengths. Regrinding of commercial cement in a vibrating mill with the addition of crude gypsum produces an excellent binder for steam treated concrete. Savings in production time and cement of increased strength can be obtained.

Variation in the strength of portland cement

P. J. F. WRIGHT, *Cement, Lime, and Gravel* (Birmingham, England), V. 34, No. 8, 1959, pp. 224-227
CERAMIC ABSTRACTS Mar. 1960

Variations in a single batch arise from unknown experimental errors in making and testing the specimens. Data are given on standard deviations in compressive strength for (1) a single batch at different ages, (2) different batches from one plant, (3) batches delivered over periods of 4 months and longer, and (4) cements produced by different plants. If the variation between batches from one plant could be reduced, less cement would be required to produce a concrete of specified minimum strength.

What do we know about plastic deformation and creep of concrete?

R. L'HERMITE, *RILEM Bulletin* (Paris), No. 1, New Series, Mar. 1959, pp. 21-51

An analysis and appraisal of 100 technical articles on creep appearing over the past 40 years augmented by the author's experimental work spanning 20 years and still in progress. Effects of composition of the concrete, method of conservation, volume under stress, age, and the magnitude and duration of loading and unloading are considered as well as creep under different stress conditions.

Among other conclusions author states that creep is an hydro-constrictive phenomenon dependent on movement of water and is co-variant with shrinkage. Deformation tends toward a value proportional to the stress with a minimum corresponding to shrinkage. Deformation is thought to follow

a law similar to that for viscosity. Rate of creep appears to decrease proportionally as the volume required increases.

Torsional vibrational experiments on rectangular concrete beams

R. Berenbaum and I. Brodie, *British Journal of Applied Physics* (London), V. 10, No. 6, 1959, pp. 281-287

Road Abstracts Sept. 1959

Three methods of measuring the tensile strengths of brittle materials including samples prepared from cement are investigated and compared with the conventional pulling method. The results obtained by the indentation test and the test in which discs are compressed diametrally, differed considerably from those of the bending test. The discrepancy is attributed to the sensitivity of the bending test to surface conditions.

Perlite concrete—Density and compressive strength in relation to mix proportions and methods of curing

F. A. Blakey and E. N. Mattison, *Report* No. C311-1, Division of Building Research, Commonwealth Scientific and Industrial Research Organization, Melbourne, 1958, 26 pp.

BRI Abstracts June 1959

The effect of four different curing treatments on the compressive strength and density of perlite concretes of differing mix proportions was studied. Results show that the strength and density of the concrete decreases with increasing perlite content and with the use of air entrainment. Unexpectedly low strengths were obtained with concrete cured in high pressure steam, and possible explanations for this behavior are noted.

Lime separation on concrete surfaces (Kalkausscheidungen auf Betonsichtflachen)

A. Menke, *Beton, Herstellung, und Verwendung* (Düsseldorf), V. 9, No. 7, July 1959

Reviewed by Ferdinand S. Rostasy

Separation of lime occurs predominantly in the cold season during which hydration progresses slower, evaporation of water slows down, and water from outside sources (rain etc.) may get in contact with the green concrete and for a long time. The concrete surfaces have to be protected against outside water. Prefabricated elements should at early age be stored in dry places. Spots due to lime separation are suitably and easily removed by scrubbing off. Acid removal should be applied with great care.

Structural Research

Testing model concrete barrel shells

P. C. Varghese, *Indian Concrete Journal* (Bombay), V. 33, No. 12, Dec. 1959, pp. 447-449

Author's Summary

Briefly describes the making and testing of a model of a barrel shell at the Indian Institute of Technology, Kharagpur. Charts compare the values of stresses obtained in the tests with theoretical values obtained by computation.

Concrete beams with tension and compression steel (Vigas de hormigon con armadura traccionada y comprimida)

Aaron Helfgot, Reprint from *Construcciones Magazine* (Buenos Aires), No. 153 and 154, 1958, 22 pp

Reviewed by Roger Diaz de Cossio

A rather complete and comprehensive report on some tests made at the laboratory of the Ministry of Public Works, Province of Buenos Aires. Sixteen beams were tested under short-time, two-point load. The variables considered were: tension and compression steel percentages, type of steel (cold worked twisted without hooks, and plain bars with hooks), and stirrup spacing. Some studies were made on the relative bond values of the two types of steel used. It was found that, insofar as strength was concerned, either ordinary or high strength steel of equivalent area can be used, and that compressive steel adds very little to the ultimate strength. When using high strength steel without a definite yield point the cracks were more evenly spaced.

Experimental flat plate structure of expanded shale concrete

Constructional Review (Sydney), V. 33, No. 2, Feb. 1960, pp. 22-29

Describes a test structure and the experimental program for a flat plate of expanded shale concrete built by the Division of Building Research, Commonwealth Scientific and Industrial Research Organization of Australia.

Encased steel stanchions and BS 449

R. F. Stevens, *Engineering* (London), V. 188, No. 4879, Oct. 23, 1959, pp. 376-377
Reviewed by Aron L. Mirsky

The 1959 edition of British Standard 449, "The Use of Structural Steel in Building," permits inclusion of the concrete encasement in calculating allowable loads on encased (fire protected) steel columns, with the allowable stress in the concrete assumed to be 1/30 that in the steel. Tests on full-size columns,* loaded axially or eccentrically, at the Building Research Station produced results which justify this; for certain eccentrically-loaded columns the new specification is well on the conservative side.

*For comments on the 1955 draft version and results of tests on small-size columns, see "Strength of Encased Steel Stanchions," F. G. Thomas, *Engineering*, V. 180, No. 4688, Dec. 2, 1955, pp. 759-760; Current Reviews, ACI Journal, V. 27, No. 8, Apr. 1956 (*Proceedings* V. 52), p. 911.

Structures of aluminous concrete (in Hungarian)

E. Bölcskei and K. Szalai, *Magyar Épitöpar*, V. 7, No. 10-12, 1958, pp. 474-477
Hungarian Technical Abstracts V. 11, No. 3, 1959

The utilization of aluminous concrete spread rapidly throughout the country between 1920 and 1930 because 1 or 2 days after placing, the structures made of this material developed an adequate load-bearing capacity. Rapid deterioration of these buildings became apparent after the war and investigations were conducted to find the causes of this phenomenon.

According to expert opinion the strength of aluminous concrete after having attained a maximum decreases slowly and gradually, especially if exposed continuously or for extended periods to high temperatures. The ultimate moments of rectangular and T-shaped cross sections made of concrete with aluminous cements were found to be lower by 14 and 4 percent, respectively, than that of specimens made of grade "B 140" concrete. Compared to the ultimate strength of the same concrete the load-bearing capacity of compressed columns is lower by 48 percent while the average surface bond strength was as low as 4.91 kg per sq cm. This reduction in the load-bearing capacity of the columns endangers the stability of the structures. A similar reduction may also be expected in structures subject to bending. A survey and registering of all structures made of aluminous concrete, observations of further deterioration, a more accurate determination of the quality of the concrete, and the strengthening of weakened columns are proposed.

General

Buying and specifying handbook

Building Construction Illustrated, V. 29, No. 4, Apr. 1960, pp. 7-116

In this issue many standard features have been omitted and the editors have assembled in compact form, certain construction information frequently wanted and frequently used. This data was selected to assist busy architects and contractors who appreciate tabulated information relative to building methods, materials, costs, etc. The following methods, materials, or applications are covered: below grade, framing, floor and roof systems, walls and partitions, reinforcing, water and vapor control, insulation, chimneys and fireplaces, building products, interior finishes, materials characteristics, building costs, and construction tools and equipment.

Each section is generally prefaced by introductory or explanatory material. This is followed by typical details and useful tables or charts.

Glossary of electronic computer terminology

ASCE Highway Division Committee on Increasing Highway Engineering Productivity, *Proceedings*, ASCE, V. 85, HW 4, Dec. 1959, pp. 89-127

This glossary which covers approximately 350 commonly used terms, provides a working knowledge of computer terminology both for the engineer directly concerned with computer program development and for those whose work requires a more general understanding of the terms peculiar to this rapidly developing field.

On various problems of geometry connected with the study of continuous media (Sur diverses questions de geometrie se rattachant a l'etude des milieux continus)

ANDRE CHARRUEAU, *Annales des Ponts et Chaussées* (Paris), V. 129, No. 4, July-Aug. 1959, pp. 375-390

Reviewed by ARON L. MIRSKY

Demonstrates certain properties of surfaces and congruences of straight lines linked to the stress tensor, and then states (without proof) the application of these to more general problems.

On shock waves (Sur les ondes de choc)

ANDRE CHARRUEAU, *Annales des Ponts et Chaussées* (Paris), V. 129, No. 5, Sept.-Oct. 1959, pp. 497-549

Reviewed by ARON L. MIRSKY

Highly mathematical study of the geometrical and kinematic transformations experienced by a continuous medium (which may be heterogeneous and anisotropic) traversed by a shock wave. An appendix contains additional material on the formulas (in matrix form) used in the study. There is a brief but well-arranged bibliography.

Architectural rendering: The techniques of contemporary presentation

ALBERT O. HALSE, F. W. Dodge Corp., New York, 1960, 277 pp., $15.75

Explains and illustrates present day techniques and media in architectural rendering. Every aspect of rendering receives attention here: interiors, exteriors, nature; perspective, lighting, reflections, textures; all of the media in detail; how to buy materials, and how and when to use them; professional tricks of the trade.

Comprehensive, detailed chapters cover graphite pencil, carbon pencil, lithographic pencil, pen and ink, smudge charcoal, Chinese ink, water color, tempera, airbrush, pastels, and combinations of media. Other sections contain a history of rendering, a study of color and light, and basic information applicable to all rendering, making the book encyclopedic in scope. Many exercises guide the user in techniques, while examples and detailed analyses of renderings in each medium give him access to expert solutions of a variety of problems.

On site precasting

Concrete Construction, V. 4, No. 7, July 1959, pp. 6-8

Describes on the site precasting of slabs and piles for a 9½ mile seawall. Slabs were cast at the rate of 50 a day and employed steel forms. This on the site casting reduced the cost of wall from $40 per lineal ft to $16 per lineal ft.

Swiss achievements

The Engineer (London), Supplement to V. 207, No. 5395, June 19, 1959, 32 pp.; also V. 207, No. 5396, June 26, 1959, p. 970

Reviewed by ARON L. MIRSKY

This supplement, a capsule (king-sized, through) review of Swiss history, achievements, current projects and major industrial plants, although aimed particularly at mechanical engineers, contains much of interest to civils: descriptions of the Grande Dixence scheme and the modified concreting procedure for the dam, the Hinterrhein scheme* and the Valle di Lei dam, and several concrete bridges, including the Weinland highway bridge.

*cf Oskar Sporri, "Hydroelectric Harnessing of the Posterior Rhine" (in French), *Le Génie Civil*, V. 136, No. 7, Apr. 1, 1959, pp. 149-158; Current Reviews, ACI JOURNAL, V. 31, No. 11, May 1960 (*Proceedings* V. 56), p. 1196.

ACI NEWS LETTER

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Taking the Institute to the Members

IN OCTOBER, 1948, the Board of Direction approved the report of a special committee on long-range planning which had studied ACI progress long and thoughtfully. Among the eight goals to be attained was the "development of a policy of *taking the Institute to the members,* such as by increasing the number of regional meetings and the holding of local conferences." Actually only one regional meeting, on a trial basis, had been held up to that time but in furtherance of this goal annual regional meetings have been held in 12 cities in the United States, Canada, and Mexico. From an initial attendance of about 100 in Birmingham in 1947, these meetings have grown in size; Montreal in 1956 drew 486 and Detroit in 1958 attracted 559 in connection with dedication of the ACI Headquarters Building.

The practice of taking ACI to the members was further extended in 1954 by the addition of a technical director to the ACI staff. He arranges local conferences on problems related to concrete and spends much time on the road during the winter attending and speaking at these meetings.

Most recent — and probably most effective — development arises from the desire of an active group of members in California for a greater exchange of technical information than was then possible. True, there had been unofficial local ACI groups which met sporadically and finally faded away. California members wanted something more permanent — more closely tied

to the national organization. Consequently, at the fall 1956 Board meeting a formal proposal for regional organization was presented by a group headed by Henry M. Layne and Samuel Hobbs. There being no formal provision in the ACI Bylaws for chapters, this group was authorized to set up and operate a provisional chapter. Official recognition as the first ACI Chapter was given to the Southern California Chapter upon adoption of revision of the Institute Bylaws at the February, 1958, convention.

In a short time two additional chapters, Northern California and Oklahoma were approved. After an informal trial period of about a year, Michigan members in 1960 applied for chapter recognition and in March officially became the fourth chapter. Currently there is considerable interest by an eastern group and one in Canada in chapter organization.

Over a period of 4 years the chapter idea has proved to be an important factor in stimulating and maintaining interest in improving concrete technology and practice. It's extension to additional local areas is a logical development which Institute officers and staff stand ready to assist.

ACI technical committee appointments

Listed below are committee members who have recently accepted appointment to ACI technical committees. Included are *new* appointments only.

Committee 115, Research

James L. Sawyer
Lone Star Cement Corp.
Hudson, N. Y.

Committee 201, Durability of Concrete in Service

Harry H. McLean
State of New York Department of Public Works
Albany, N. Y.

Thomas J. Reading
Missouri River Division
Corps of Engineers
Omaha, Neb.

Joseph J. Waddell
Knoerle, Graef, Bender and Associates, Inc.
Chicago, Ill.

Committee 325, Structural Design of Concrete Pavements for Highways and Airports

L. D. Childs
Portland Cement Association
Skokie, Ill.

W. T. Spencer
State Highway Department of Indiana
Indianapolis, Ind.

W. M. Stingely
Kansas State Highway Commission
Topeka, Kan.

Committee 331, Structures of Concrete Masonry units

J. H. MacDonald
Supercrete Limited
St. Boniface, Manitoba, Canada

Harry Mitchell
Spartanburg Concrete Co., Inc.
Spartanburg, S. C.

O. Neil Olson
Marquette University
Milwaukee, Wis.

Thomas J. Reading
Missouri River Division
Corps of Engineers
Omaha, Neb.

Edwin L. Saxer
University of Toledo
Toledo, Ohio

Walter Washington
Summer Sollitt Co.
Chicago, Ill.

Committee 333, Design and Construction of Composite Structures—Joint ACI-ASCE

George C. Driscoll, Jr.
Lehigh University
Bethlehem, Pa.

Committee 336, Combined Footings

S. V. DeSimone
Moran, Proctor, Mueser and Rutledge
New York, N. Y.

William C. Siler
Jackson and Moreland, Inc.
Boston, Mass.

Ingvar Schousboe
Civil Engineer
Crystal Lake, Ill.

Committee 402, Concrete Floor Finishes

Herbert K. Cook
The Master Builders Co.
Cleveland, Ohio

Committee 505, Design and Construction of Reinforced Concrete Chimneys

Nelson M. Isada
University of Buffalo
Buffalo, N. Y.

Committee 609, Consolidation of Concrete

R. W. Thornburg
Blaw-Knox Co.
Mattoon, Ill.

Committee 714, Concrete Bins and Silos

Edward D. Baxter
A. E. Baxter Engineering Co.
Buffalo, N. Y.

McGinnis to represent ACI on graphic arts committee

Howard J. McGinnis, Structural and Railways Bureau, Portland Cement Association, Chicago, is serving as ACI representative on the Task Group on Building Facilities, Committee on Engineering and Materials Handling, Research and Engineering Council, of Graphic Arts Industry, Inc., which was organized to develop recommended practices for flooring in graphic arts plants.

Mr. McGinnis is currently chairman of ACI Committee 402, Concrete Floor Finishes, and a member of ACI Committee 401, Specifications for Structural Concrete.

ACI cooperates in thin shell lectures in Cambridge

Over 100 engineers from the Boston-Springfield-Providence area have attended a series of twice-weekly lectures on the analysis and design of thin shell roofs, sponsored by ACI and Sumner Schein, architect and engineer. The lectures were held in Cambridge, Mass., beginning on March 23 and running through May 25.

Among the topics discussed were: membrane and bending analysis of domes, cylindrical shells, conoids, hyperbolic paraboloids, and arbitrary surfaces, folded plate roofs; ultimate strength design of shells; and computer analysis of shells.

The lectures were given by Howard Simpson, member of the consulting firm of Simpson, Gumpertz and Heger, Cambridge. Dr. Simpson recently resigned his position as associate professor of structural engineering at Massachusetts Institute of Technology to devote full time to his consulting practice.

Dean Kerekes awarded ASCE life membership

ACI past president Frank Kerekes, dean of the faculty, Michigan College of Mining and Technology, Houghton, has been awarded a life Membership certificate in the American Society of Civil Engineers. The presentation was made at a special meeting held May 13 in Houghton, Mich.

This represents only one of many awards bestowed on Dean Kerekes in recognition of his contributions to the field of engineering. He has devoted his life to education and engineering. He served 34 years on the civil engineering faculty at Iowa State University, Ames, and advanced to assistant dean of engineering before taking over his present role at Michigan College of Mining and Technology.

An ACI member since 1938, Dean Kerekes has served the Institute in many capacities on executive and technical committees. He was president of the Institute in 1956 and is currently a member of Committee 318, Standard Building Code, of which he was chairman, 1950-56.

2000 A. D.—
Construction Challenge

WHOLE TOWNS DELIVERED BY AIR, large buildings transported by rocket, cities that are disposable, the growth of 14 megalopolises in the United States, houses shipped C.O.D., and "seed" structures that blossom into huge buildings.....

These are some of the marvels foreseen for the world of 2000 A. D.

What will it be like? There are as many predictions as there are predictors — a range from the wildly fantastic to ultraconservative.

Age of the megalopolis

Webster defines megalopolis as "a very large city." Megalopolises will be one of the most striking characteristics of the final quarter of this century.

There will be 14 of them in the United States. Some will be as much as 600 miles long and 100-200 miles wide. Already they are taking shape.

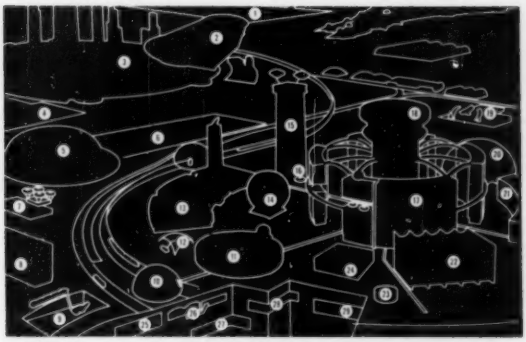
A typical one extends 450 miles from Boston to Washington, D. C. Most, naturally, will be in the East. There will be four in the West. Small ones around Denver and Salt Lake City; big ones running from Seattle to Salem, from Sacramento to San Diego.

Strip city is a name which has been attached to this phenomenon.

The cities of today won't disappear because of this, they will actually become the cultural, economic, and commercial centers of the "strips". The accompanying drawing illustrates many of the future-city characteristics of this "heart" of the megalopolis.

This article was abstracted from *This Earth*, Permanente Cement and Kaiser Gypsum Companies, V. 12, No. 1, Jan. 1959.

guide
to
drawing



PICTORIAL impression of the city of the future is a composite embodying many of the building designs and patterns of work and leisure which will be commonplace by 2000 A.D. Industrial plants (1) will be clean, attractive and occupy designated areas on the fringe of the city. Flying platforms, or "saucers," will have a key role in personal transportation and in the construction industry. One function will be air delivery of factory-built homes (2). Structures with related purposes will be grouped in circular patterns. One of the city's business centers is depicted at upper left (3). Orderliness, beauty, landscaping are the keynotes.

One trend in housing will be inexpensive, concrete row housing (4) which will emphasize livability, privacy, and surrounding parks. Sports will be a vital part of man's expanded leisure time. Facilities will be expansive and close-in to the city-heart. Multi-purpose stadia will have retractable roofs (5) and will be adjoined by participator-sports facilities (6). Flying platforms will also be used to deliver giant precast building sections (7) from factory to site. Near a large public hospital (8), excavation work for the foundation of a new building (9) is carried on by an automatic excavator. Earth is carried by conveyor to waiting train of huge earth moving buggies. Surface transportation within the city will be by suspended monorail trains. Monorail stations (10) will be concrete bubble domes.

Culture and government will hold forth in the center of the city — again with the buildings arranged in a circular pattern. In the outer ring will be a heliport (11) for arriving and departing airport passengers. To its left is an auxiliary structure (12) which is being finished with a poured-in-place structural gypsum roof. Next is a sprawling communications center (13) which handles the city's telephone operations, as well as housing all the radio and television stations. A ball-shaped recreation center (14) is topped with a glass-enclosed public swimming pool. Behind it, a towering slip-form structure is under construction (15). Concrete is being delivered to the job by special flying platforms.

Near the structure's base, a passenger platform (16) is about to land in one of the mid-building "ports" of the governmental center (17) — the core of the city heart. This intricate network of four curved office buildings forming a broken circle around a barrel-like central building houses all the metropolis' governmental operations — municipal, state and federal. The struts connecting the buildings are enclosed walkways. Topping the circular building (18) is the center's own heliport. To the right rear of the governmental center is a solar power plant (19) which supplements the city's nuclear power supply. Other buildings comprising the city heart are a concrete geodesic dome civic auditorium (20); a library and science museum (21); a theater (22); restaurant (23); and art museum (24). Cutaway section in foreground reveals some of the city's underground facilities: parking garage (25); train station (26); nuclear power plant (27); water purification and re-circulation plant (28), which enables city to re-use indefinitely the same water; and a sewage disposal plant (29).

Structures of tomorrow

The predictions of R. Buckminster Fuller, developer of the geodesic dome, holds special interest:

"Structural accomplishment by 1975 will find man occupying, mining, and developing both of the polar regions as well as all the now-hostile tropic, desert, and mountain areas. In addition, he will be occupying large areas of the ocean floor.

"Rocketry of large buildings, explosively transforming into full size and operable conditions at the target, will be commonplace. Air deliveries of whole towns by parachute drop will be a usual occurrence."

Construction's key to tomorrow: Prefabrication and automation

Construction will be "big business," will be widely conducted on an assembly-line basis; annual volume will exceed $100 billion a year.

The future of heavy construction can already be seen in the well established trends toward more speed and economy to fabrication and radical new uses of materials.

The construction industry already has put to work the modern marvels of cybernetics, aviation, television, electronics, atomic physics, and automation.

But these are the "Model-T's" of tomorrow. What will they lead to by this century's end? The experts are broadly unanimous:

- Real manual labor—hand craftsmanship in construction—will be virtually banished.
- Structures will be assembled—not built piece-meal on site—in large sections molded in prefabricating yards.
- The big jobs—excavating, paving, lifting, moving—will be done by giant machines. Functions will be combined.
- Engineering advances and materials technology will permit tremendous flexibility and freedom of imagination in design.
- Projects of a size and complexity unknown today will be on the books.

The machines

Interesting developments in the application of machinery to construction are due. It is likely that full scale mechanization will bring on a new look, both inside and outside the construction industry.

If prefabrication is to be the order of the day, as the experts claim, then there must develop a far more organized and standardized construction industry. It's likely that the building industry will develop to the point that it can truely call itself an industry in the same sense as do today's automobile manufacturers. Bigness — corporate giantism—will become a necessity, steming from the huge investments that will be required to establish research, development, production, and distribution of prefabricated homes and buildings.

The major components of homes and commercial buildings will come off assembly lines just as cars do today. Partitions, floors, curtain walls, structural members, roofs, and utility service cores will be factory produced.

Assembly on the site will have progressed to a point of mechanization where large repetitive jobs, such as curtain wall placement, will be performed by automatic machines directed by operators from control stations.

Methods and materials

It's a good bet that most of the basic materials of today will be much in evidence in 2000 A. D., but in hundreds of new forms, shapes, combinations, and refinements.

Some engineers believe that the portland cement industry will, before the century closes, have developed a radically new product to create concrete-like structures with higher tensile, plastic-like strength. This new concrete would expand and contract less with temperature changes and offer great durability.

Of whatever chemistry, concrete will really come into its own by 2000 A. D.

Prestressed concrete will lead the way. Arthur Anderson of Tacoma's Concrete Technology Corp. noted that "in establishing the criteria for an ideal material of construction we seek something that is: strong and durable, economical, fireproof, corrosion-resistant, rot-and-termite proof, readily available, easily molded into a variety of forms, and architecturally satisfying.

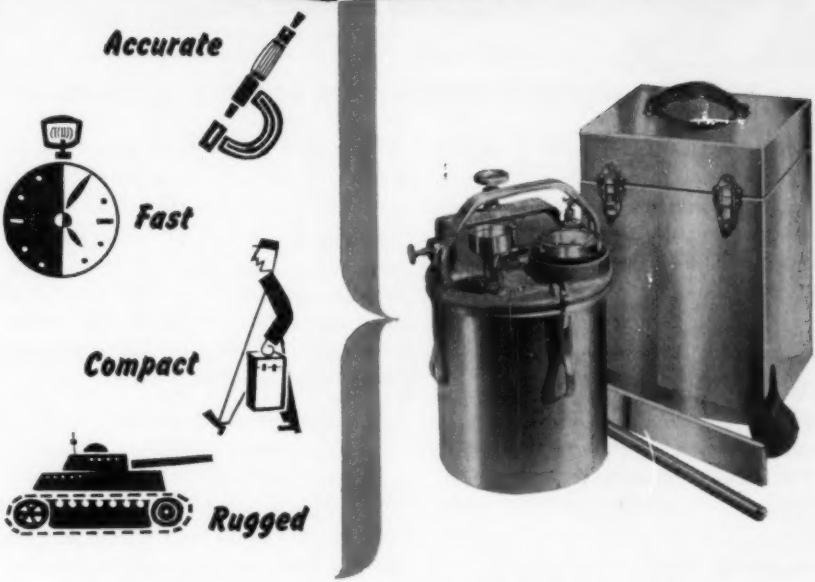
"After consideration of all currently known materials on all eight qualities above, prestressed concrete obviously scores highest." Dr. Anderson also predicted concrete strengths will advance to 15,000 psi.

Professor T. Y. Lin of the University of California feels that prestressed concrete will become increasingly useful

proof of performance makes

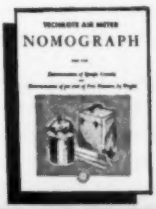
TECHKOTE AIR METER

the world's leader



The TECHKOTE AIR METER gives by far the most dependable accuracy obtainable—and does it for the reason that the unit itself is precision-built from start to finish. The extremely close tolerances combined with rugged construction assures long lasting accuracy both in the laboratory and field. Simplified design makes for quick operation and easy maintenance.

The Air Meter combined with the outstanding and exclusive TECHKOTE NOMOGRAPH gives a highly diversified and complete concrete engineering unit.



A-M

PRESSTITE DIVISION

AMERICAN-MARIETTA COMPANY

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Darex Diary

by Mel Prior
Member, American Concrete Institute
Technical Service Manager, Construction Products
W. R. GRACE & CO.
Dewey and Almy Chemical Division

(No. 19 of a series)

AVAILABLE MATERIALS FOR CONCRETE FLOOR REPAIR

FREQUENTLY I am asked if there is any simple way to repair concrete floors or change the grade for drainage purposes. There is no simple method of accomplishing this, but there are materials available which make the job easier and more certain of success. These materials may be divided into two general classes.

One class is the epoxy resins. This material is not used as an admixture, but as a bonding or surfacing type applied directly to the surface requiring repair. It is furnished as a two component material which combined in proper proportions, results in an extremely strong bonding agent in a matter of minutes. Since the pot life of this material is limited, it must be made in relatively small batches and used immediately. Some successful applications have been made using bulk quantities on a large scale, but at the present time the high cost of epoxy has been a deterrent to its use.

A second class of material, the emulsion types of bonding agents, may be subdivided into two classes: (1) natural and (2) synthetic. The natural latex emulsions are critical because of their general tendency to coagulate in the presence of alkalis. Hence it is necessary to use pre-tested sand and cement with this type of agent. The synthetic polymers may, in general, be used with any suitable materials and are, therefore, more popular. The pot life of these materials when mixed with cement is basically the same as that of portland cement when mixed with water.

The common types of synthetic emulsions are homopolymers, commonly referred to as PVA's. These materials have the property of being re-emulsifiable. They will soften, resulting in loss of bond in the presence of water, hence are not suitable for application where moisture is a factor.

Other materials, such as copolymers, do not lose bond in the presence of water and can be used in exposed locations. DARAWELD is one such material. When added to a portland cement mix, it increases the adhesive properties of the mix to such an extent that it will stick to any clean surface, absorbent or nonabsorbent. DARAWELD is a synthetic material and is compatible with any type of portland cement and/or aggregates that are suitable in the manufacture of concrete.

W. R. GRACE & CO.
DEWEY AND ALMY CHEMICAL DIVISION

Cambridge 40, Mass.; San Leandro, Calif.; Chicago 38, Ill.; Montreal 32, Canada

in construction of thin-shell roofs, long and thin roof sections spanning large open expanses, and in slabs, panels, and columns. Prestressed, clear-span structures up to 1300 ft have been designed. Dr. Lin also noted that the technique was ideal for prefabrication.

Self-stressing concrete

A highly promising offshoot of prestressing, but still in the laboratory and discussion stage, is self-stressing concrete. It is being researched by the University of California and by French and Soviet technicians. The following is extracted from a Russian report:

"Research conducted at the Central Industrial Constructions Research Institute has resulted in the creation of a cement named "stressing cement" which, due to its chemical composition and methods of hydrothermal treatment in the process of placing and hardening, increases the volume and stresses all the bars of the reinforcing framework, irrespective of their direction in the concrete.

"Stressing cement is formed as the result of dry-grinding of portland cement with aluminoferrous cement and gypsum. The stressing cement sets quickly, within 2 to 5 min.

"These resulting problems were successfully solved by applying shotcrete work, which provides for an initial preparation of a dry mixture of sand and cement; this mixture is moistened at the moment of leaving the nozzle and in a fraction of a second hurled onto the surface of the cast article.

"The perfecting of stressing cement, the mastering of its technology, and the designing of new constructions call for serious collective effort on the part of scientists and engineers."

Variety of roles

Because it is a plastic, and a low-cost one at that, concrete will fill a tremendous variety of roles in future construction. There will be great expansion in the following applications:

- Lightweight concrete, that floats and can be sawed by hand, is already a reality.
- Pneumatic applications, such as shotcrete, will become commonplace.
- Foamed (cellular) concrete, to fill inflatable forms that later are peeled away, is a definite possibility.

These last two methods might well be key components that will help fertilize R. Buckminster Fuller's concept of "seed" structures. He says: "Air-deliverable structures will obviously consist of alloys of extraordinarily high performance - per - pound characteristics. They will be designed in mathematically foldable techniques, permitting expansion ratios in the magnitude of an acorn's expansion to an oak tree. These delicate structures will frequently be skinned and fireproofed with concrete, due to economic advantages inherent in the availability of cement and aggregates."

Prestressed concrete

Likewise, Fuller foresees a large role for prestressed concrete in his geodesic structures: "The prestressed, alloy-reinforced concretes (as well as the nonprestressed) will be in enormous use as larger and larger structures develop. The minimum effective sectional dimensions of prestressed concrete components will, when employed in geodesic domes which we are now able to compute in 1-mile diameters, seem ephemeral as scaled to the whole structure.

"Key to these giant, clear-span undertakings, which can put whole cities under cover in the arctic, tropics, and under-sea is the discontinuous compression, continuous tension structure, in which the larger the structure—the more diffused are the local working loads and stresses. In the very large structures, the local compressional components may be as freely and locally treated as, in the same way, the earth's surfaces itself—which integrity is entirely gravitational — i. e. tensional."

Frequently mentioned in builders' predictions are such relatively new techniques as slip-form and lift-slab construction, except they would involve whole buildings of considerable complexity. Both are close in spirit to prefabrication.

Mainly, though, they are indicative of the trend to "engineered" construction, and point the way to economical building of the future's fabulous designs.

More predictions

In road building and excavation—earth moving of all kinds—field engineering will have to keep pace with faster and larger machines.

New forms of building will be seen everywhere, such as huge arches, vaults, scallops, angles, and so on. Many of these forms are the direct result of the advent of prestressed thin-shell concrete.

The Noble Co. of Oakland, engineers and manufacturers of concrete batching systems, previewed its industry as follows: "Automation of concrete plants is keeping pace with automation of all types of industrial equipment. Already we have automatic plants which are remote-control operated by the dispatcher. As many as 100 pre-set batches and mix proportions can be selected at will by the push of a button. The next obvious advance will be a plant which can be radio-controlled from incoming mixer trucks. In this manner, a batch of preselected mix proportions will be weighed and ready as the truck pulls under the plant.

"Because of the need for more production capacity, high-speed turbine type concrete mixers can produce concrete at more than twice the rate of the older drum type mixers of comparable size.

"The ideal plant would eliminate the undesirable features of high elevation and intermittent batching which are expensive and time wasting. Our goal is ultimately to develop a low-height continuous plant, producing high output, in which materials are preblended (rather than batched) in exact proportions."

Today, the construction industry stands of the threshhold of a whole new world . . . a world so new it will bear only a slight resemblance to that which we know today. Whether we agree or not with some of the predictions, there is no doubt that the world we live in will be reshaped considerably by construction's accelerated technology.

AN APPRECIATION
Harry F. Thomson

With profound sorrow, the Board of Direction of the American Concrete Institute mourns the loss of Harry Freeman Thomson by death on Nov. 7, 1959. For more than 30 years, Mr. Thomson was active in Institute affairs as author of technical papers, member and chairman of committees, and officer of the Institute, climaxed by his service as President in 1951. He was a member of the Board of Direction in 1939 and 1940 and again from 1942 to 1948, and was vice-president in 1949 and 1950. He maintained his interest in ACI to the last, and at the time of his death he had just attended the regional meeting in Mexico City.

Harry Thomson's high professional ability and ethical standards, and his devotion to the aims and activities of ACI, have long made significant contributions to Institute progress. His warm and friendly personality will be missed by his associates on the Board of Direction of the Institute which he served so well.

BOARD OF DIRECTION

Lakeview Elementary School used concrete masonry for exterior and interior walls. The school was erected in 5 months at a cost of $10.91 per sq ft

Low-Cost Concrete Masonry School

ECONOMICAL CONSTRUCTION was uppermost in the mind of San Diego architect George Lykos when he designed the all concrete masonry Lakeview Elementary School in Lakeside, Calif.

From the architect's standpoint, the knottiest problem was to capitalize on the inherent values of concrete masonry in a structure that could be built at a per-square-foot cost comparable with the ordinary school buildings in San Diego County.

"We solved the problem by working for simplicity of all structural connections and ties," Lykos said. "We worked in the direction of taking construction man-hours out of the project—of making work simple for the mason."

He said that his firm began by working on a simple design based on the modular system of the individual block, 8 x 4 x 16 in. After designing a plan that would permit simplest masonry work, they simplified structural details to permit easy placing of steel and similar man-hour-saving construction techniques.

The result was a 15,065-sq ft school building erected at a cost of $164,480—about $10.91 per sq ft. This is only about $1 per sq ft higher than a comparable frame structure. The extra initial cost, however, is offset in the lower maintenance expense of the concrete masonry building.

The Lakeview school was built in 5 months. It contains 12 classrooms positioned "back-to-back" so that all classrooms have access to the outside and have windows the full length of one wall. Clerestory windows provide for ventilation. The concrete block walls have been made high enough so that noise is prevented from penetrating from one classroom to another.

Outcropping concrete block made simple attractive wall pattern. Outcropping block are 1 in. wider than standard 8 x 4 x 16-in. wall block

The school is painted brown on the exterior, and in yellow and multitone tints on the interior. Interior walls are exposed masonry block, with the paint applied directly to the block.

Approximately 25,000 Rocklite concrete blocks, supplied by Hazard Products, Inc., San Diego, were used in the structure. The walls are solid-filled with concrete grant, as required by the State Division of Architecture, eliminating all voids.

Prime contractor on the Lakeview school project was J. P. Bender Co., San Diego. Masonry contractor was Wallace Masonry Co., El Cajon, Calif.

LOOKING AHEAD

July 6-8, 1960—Cooper Union-ASCE-ASEE Conference on Civil Engineering Education, University of Michigan, Ann Arbor

July 10-22 and July 24-August 5, 1960—Engineering Seminars on Atomic Shelters, Pennsylvania State University, University Park, Pa.

July 11-18, 1960 — Second World Conference on Earthquake Engineering, Science Council of Japan, Tokyo and Kyoto, Japan.

July 17-21, 1960—RILEM International Symposium on Concrete and Reinforced Concrete in Hot Countries, Israel Institute of Technology, Haifa, Israel

July 18-22, 1960—Thin Shell Concrete Structures Study Course, Case Institute of Technology, Cleveland, Ohio

Aug. 15-26, 1960 — Summer Institute on Nondestructive Testing, Sacramento State College, Sacramento, Calif.

Aug. 29-Sept. 2, 1960—Symposium on Reinforced Concrete Design, Queen's University, Kingston, Ont., Canada.

Sept. 27-30, 1960 — 6th Annual Convention, Prestressed Concrete Institute, Statler-Hilton Hotel, New York, N.Y.

Oct. 10-13, 1960—American Mining Congress, Metal Mining and Industrial Minerals Convention and Exposition, Las Vegas, Nev.

Oct. 31-Nov. 1-2, 1960—13th Regional Meeting, American Concrete Institute, Pioneer Hotel, Tucson, Ariz.

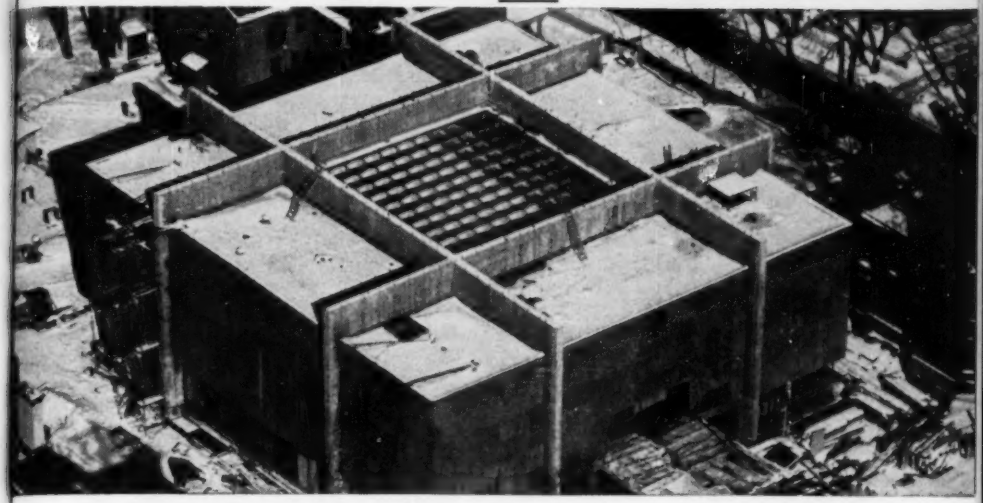
Nov. 3-4, 1960—Annual Meeting, National Slag Association, Hotel Mayflower, Washington, D.C.

Dec. 12-15, 1960—First Industrial Building Exposition and Congress, New York Coliseum, New York

Jan. 5-7, 1961 — FIP-RILEM Symposium on Injection Grout for Prestressed Concrete, Norges Tekniske Hogskole, Trondheim, Norway

Jan. 9-11, 1961—Annual Convention, Concrete Products Manufacturers Association of Quebec, Chateau Frontenac, Quebec, Canada

POZZOLITH . . . makes good concrete better



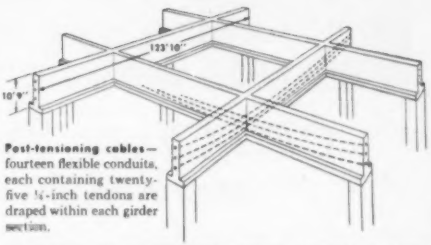
Unique structural system for Utica's newest art gallery

The huge monolithic, intersecting prestressed girder system shown here is the heart of a unique engineering and construction achievement. In a continuous 16-hour placing operation, the girder was cast in place fifty feet above grade. Dr. Lev Zetlin's mix design included POZZOLITH to provide concrete meeting these rigid requirements:

- 28-day strength of 5000 psi.
- Slow initial set to avoid cold joints.
- Extremely high degree of workability to prevent voids and honeycombing in girders crowded with reinforcing steel and post-tensioning cable conduits.
- Lowest possible unit water content.

In addition, POZZOLITH was used in all job-placed concrete—for foundations, walls and exterior supporting columns to obtain increased workability and minimum shrinkage.

The local Master Builders field man will welcome discussing your requirements. Remember, POZZOLITH concrete is higher in quality and more economical than plain concrete or concrete made with any other admixture.



Post-tensioning cables—fourteen flexible conduits, each containing twenty-five ¼-inch tendons are draped within each girder section.

DESIGNED BY ARCHITECT PHILLIP JOHNSON AND ENGINEERED BY DR. LEV ZETLIN—this concrete tic-tac-toe provides an interior free of columns for Munson-Williams-Proctor Institute Art Gallery in Utica, New York. Contractor: George A. Fuller Company, New York; Supervising Architects, Bice and Baird of Utica. POZZOLITH ready-mix concrete from American Hard Wall Plaster Company, Utica.

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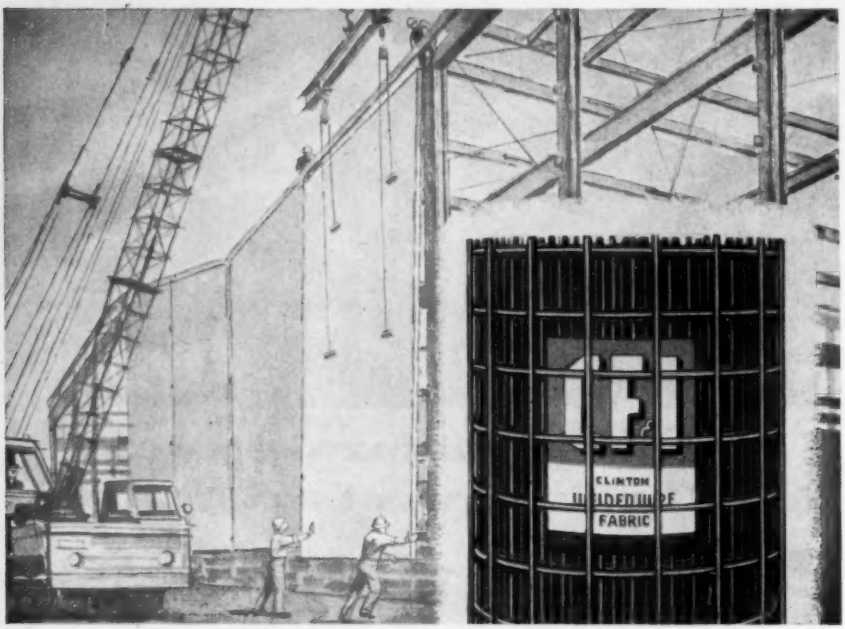
MASTER BUILDERS® POZZOLITH*

**POZZOLITH is a registered trademark of The Master Builders Co. for its concrete admixture to reduce water and control entrainment of air and rate of hardening.*



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Silhouetted against the sky's last light of a short wintertime Maine day, workers rush to complete network of structural and reinforcing steel for walls of U. S. Navy's new transmitter building. On scaffolding, welder is brazing "jumper" between mat of heavy welded wire fabric, and lintel beam over entrance. Electronic requirements of transmitting equipment call for positive grounding of all metal in building, with no loose metal-to-metal contact. At bottom, right, worker is cleaning reusable steel forms for next day's concreting

Naval Transmitter Building Presents Unusual Problems

"UNUSUAL" IS THE WORD for the structure built in Cutler, Me., to house the transmitter for the new Naval Radio Transmitting Station—to be the most powerful in the United States. The 25,000 sq ft reinforced concrete building presented unusual requirements to designer and builder alike, and they were met with unusual solutions.

Charged with the design of the transmitter house (and for all other physical facilities except actual transmitter equipment) were Hayden, Harding, and Buchanan, Inc., Boston, and James W. Sewall Co., Old Town, Me., in joint venture.

Wall thickness

One of the first problems to be considered concerned thickness of the reinforced concrete walls specified by the Navy. Both interior and exterior surfaces of the walls were required to be flush, with no offsets for structural columns being permitted. Furthermore, the 25 ft high walls, rather than just enclosing space, were to be partially load bearing, taking some of the roof load between columns, the roof being planned as a two-way slab.

Although a 10 in. thick reinforced concrete wall would have been sufficient to encase the steel columns completely, and would have been structurally safe, a 12 in. minimum thickness was settled on. It was reasoned that the greater thickness would materially ease and speed up placement of concrete between the forms and a double line of reinforcing. In addition, it would give added stiffness to the panels between the 15 ft spaced columns. Furthermore, the added thickness would ease the load on the air conditioning system which will be installed to keep the temperature and humidity of the transmitter operating areas within acceptable limits.

Reinforcement

Given wall thickness, the selection of minimum temperature steel reinforcement normally is a matter of simple computation. But another unusual

requirement now faced the designers. With large inside areas of the building later to be completely shielded with copper lining, floors, ceiling and walls —all other metal in the structure had to be electronically and positively grounded.

This stringent requirement caused the designers to select mats of heavy welded wire fabric for reinforcement. In accordance with the ACI Code, they specified two lines of 6x6-0/0 fabric, furnishing 0.296 sq in. of steel area per foot in each direction.

Two reasons were given for this choice. (1) Because of the electronic grounding requirement, there could be no loose or uncertain metal-to-metal contact; therefore each intersection of reinforcing members had to be welded. Rather than call for individually placed and welded reinforcing bars, the engineer selected 8 x 10-ft mats of fabric. With the mat's 357 intersections of 6 in. spaced wires electrically fused at the factory, an obvious saving of many on-the-job welds was anticipated.

(2) Because the mats of fabric offered a uniform face to work on, with each intersection the same as the other, it was felt that they would offer a better clearance between the wall forms and save further handling time of reinforcing steel.

Grounding requirements

To complete the requirement that all metal in the transmitter building be electronically grounded, the design firms specified that cross wires of each mat of fabric be welded at all laps between sheets, on all four sides. This, naturally, had to be done on the job. Where mats crossed columns, a weld was required each 4 ft. Further, brazed "jumpers" between inner and outer lines of fabric were specified at intervals to assure positive electronic connection.

Similar specification for welding and electronically grounding the lighter mats of fabric (6x6-2/2) for reinforcement of the inner, nonbearing 9-in. thick partitions were established. Even the 6x6-6/6 ground slab wire reinforcement had to be grounded, as did the 4x4-10/10 with which the columns were wrapped for better bond of their concrete encasement.

Finally at the exterior wall of the building, around its entire periphery, were driven at 5-ft centers, 5 ft long copper clad steel rods, to which was connected by copper busses the entire structural and reinforcing steel system. Even steel lockers and toilet partitions are to be grounded.

Radiating out from the building and connecting to the copper bus is a buried network of copper wires connecting to the antenna-ground system, which completes the electronic grounding circuit for the structure.

Architectural treatment

Esthetic requirements governed the designers too. Only a handful of naval people are expected ever to view the transmitter building. Nevertheless, the broad expanses of 25 ft high concrete wall promised to be pretty drab, and challenged the designers for relief.

The designer's answer was to call for vertical ribbing of the walls. by the simple expedient of inserting corrugated cement asbestos sheets next to the outside forms. This satisfied esthetics, but passed a problem on to the building contractor.

Construction teamwork

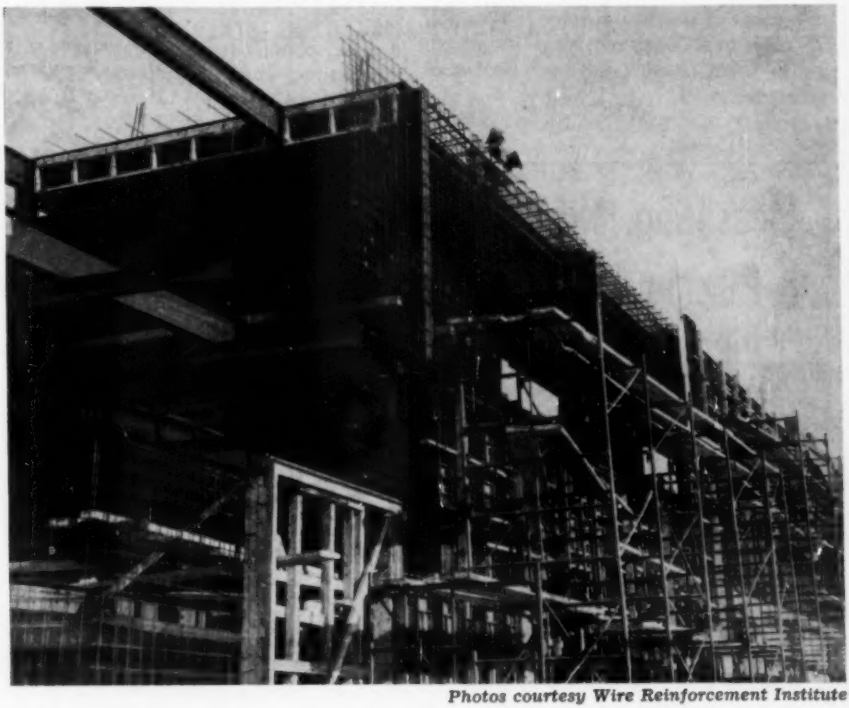
Best measure of how the contractor, Consolidated Constructors, Inc., New York and Portland, Me., handled the unusual requirements of the transmitter house is found in the following description. Despite unusually bad weather—several consecutive weeks of subzero temperatures—the contractor has kept the job moving, using labor- and time-saving methods and equipment.

In handling the 8 x 10-ft mats of fabric, two men readily positioned the lower reinforcing, with a crane being

used to hoist the mats to the scaffolding for upper courses of the wall. One welder, working 15 min, tacked a sheet into position on the side and bottom. Consolidated's engineer-superintendent, Curt Snow, estimated that placing and fastening a comparable amount of individual reinforcing bars would have required about 3 hr.

Taking advantage of the speedy operations of placing, fastening, and grounding the fabric, the contractor was able to move more rapidly into forming, concreting, and quicker re-use of forms, all of which helped account for time saved on the job.

Good teamwork also helped Consolidated to get ahead of the clock. Following on the heels of the welders, workers closed up the forms around the two lines of steel reinforcement. Using Econo-Forms, reusable steel forms in several sizes, based on a module of 2 x 4 ft, the contractor usually placed two tiers at a time, with the wall being cast in 8-ft lifts. Before closing the outside forms, 42 in. wide x 8 ft long sheets of corrugated cement asbestos board



Photos courtesy Wire Reinforcement Institute

With double lines of heavy welded wire fabric mats in place for temperature reinforcement of Naval transmitter building exterior walls, workers (on roof) are placing right angle bars which tie fabric to roof reinforcement. At far right, several bays of wall have been concreted and protective tarpaulins are visible at top of wall. The reusable steel forming has progressed to second column to right of workers and is about half way up on the two bays in the foreground. In the bottom left the double rows of reinforcement have been bent to form a corner of the building

were positioned, to form the vertical ribs in the exterior walls.

The cement asbestos was fastened back against the steel forms with several wire ties—sufficient to hold it in place until it received the thrust of the fresh concrete.

The cement asbestos board at first was a problem to work with. After stripping the forms, it was found difficult to break away the board from the concrete. The solution was to coat the cement asbestos with Form-Film, a lacquer type bond breaker usually used for plywood. With the use of the film, the pores of the board were sealed and a glass-like surface was imparted, which resulted in a superior wall surface.

Measuring, Mixing & Placing Concrete

RECOMMENDED PRACTICE FOR MEASURING, MIXING AND PLACING CONCRETE (ACI 614-59). **This ACI Standard outlines good practices for measuring and mixing the ingredients for concrete and for placing the finished product. Practices designed to achieve maximum uniformity, homogeneity, and quality of concrete in place are given special consideration. Recommendations cover handling of aggregates, batching procedures, mixing requirements, handling of mixed concrete, methods of placing concrete in forms, and consolidation. 32 pp. $1.00 per copy, 50¢ to ACI members.**

AMERICAN CONCRETE INSTITUTE **PUBLICATIONS**

While the forming was going on, other workers stripped forms from the placement of a few days previous, and cleaned and oiled them with Corvus bond-breaking oil for later re-use. Concreting went on bay-by-bay, lift-by-lift, except when the temperature fell below 10 F. Using 100 F water and heated aggregates, the ready-mixed concrete plant operator, G. E. Goding, supplied Consolidated with concrete by transit-mix truck. Other than Darex air-entraining agent, no admixtures were used. Concrete for the building was specified as 3000 psi at 28 days, with a 1 in. maximum aggregate below grade.

The sections cast were covered with tarpaulins, and gasoline space heaters used to build up to 70 F under the coverings for 3 days—and 50 F for the next 4 days.

All construction joints in the 12 in. thick wall were keyed for strength and continuity, both vertical joints and horizontal joints between the 8-ft lifts. In each type of joint, the lines of wire fabric reinforcement were "pigtailed" out of the previous placement and continued through the joint without a break.

Another instance of the devices used by the contractor to save money and time was his use of Spanall adjustable trusses. The use of these steel units eliminated costly, time-consuming shoring and underpinning of the roof formwork.

Although Consolidated Constructors, Inc., has already completed its work on the transmitter building, there remains much work to be done before the new facility goes on the air in January, 1962. Hayden, Harding and Buchanan, Inc., is presently doing architectural and engineering work for a fuel pier, and fuel facilities, while preliminary engineering and field studies in connection with the administrative area, high frequency area, and housing area also are under way.

Credits

Hayden's chief structural engineer is Robert Parker. Other contractors involved in the transmitter building work are: Kerr Electric Co., Auburn, Me.; C. H. Babb & Co., Bangor, Me., plumbing, heating and ventilation; Bangor Roofing Co.; Mooers Construction Corp., Gardner, Me., excavation and subgrade. Barrcroft and Martin Rolling Mills, Portland, Me., furnished all reinforcement and structural steel. Responsibility for electronic design and performance of the transmitting station lies with Continental Electronics Manufacturing Co., Dallas, Tex.

First Naval District (Boston) public works officer in charge of construction is Capt. A. C. Husband. CECUSN. Resident officer in charge of the transmitter project is Capt. J. R. Burke, CECUSN, with Commander John Richardson, CECUSN, as first assistant, and Lt. L. B. Taylor, CECUSN. second assistant.

Dipping Speeds Form Coating

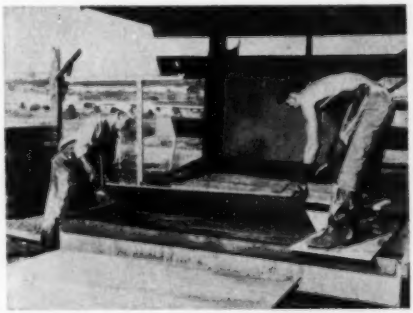
A dipping technique to coat fir plywood concrete form panels was developed by Engstrum & Nourse, San Francisco building contractors, to help speed construction of dormitories for San Francisco State College. The buildings total 127,000 sq ft of area, and required 8000 cu yd of concrete for two six-story structures.

The system can be used either in a contractor's yard or on a job site. An 18-gage trough of galvanized steel, 5 x 10 x 1 ft deep, was set up on the building site under a rough sheltering roof. At the rear of the trough was a sloping metal shelf equipped with three notched stringers which supported up to 25 panels.

Engstrum & Nourse used ¾-in. and ⅝-in. B-B PlyScord in 4 x 6- and 4 x 10-ft sizes. Two men worked at the dipping trough. It first was filled to a depth of several inches with Burke form sealer dipping formula. Then panels stacked in front of the trough were dipped one at a time and allowed to soak up the solution for as long as 3 min. (Brushing panels required up to 15 min each and did not produce as thorough a result, particularly on the edges.)

After dipping, the panels were stacked to drain on the rack—the drippings running back down into the trough. At the end of a run, the panels were restacked horizontally in an adjacent pile, and stickers inserted to permit all surfaces to dry. By the following morning they were ready for repiling without stickers, and ready to use on the job.

On this job, the same panels were re-used for all six levels of the buildings.



Two men coat a plywood panel with concrete form sealer in 3 min by dipping the panel in the galvanized steel trough containing the sealer. After dipping, the panels are stood on edge on the rack in the background with the drippings running back into the trough. The panels were then stacked separated with sticks to dry out ready for use the following day

Precast Concrete Components Lifted and Placed with Richmond Lifting Inserts

Precast wall slab being tilted into place by means of Richmond Lifting Inserts and Lifting Brackets.

Certified tests by an independent laboratory assure proper strength rating for efficient design and performance in concrete at usable strength

Richmond has developed and tested a complete line of Lifting Inserts and accessories for handling and placing of precast concrete wall slabs, columns, beams, girders, piles, etc.

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These units are designed with adequate extra strength and for simple operation in lifting and bracing precast concrete components.

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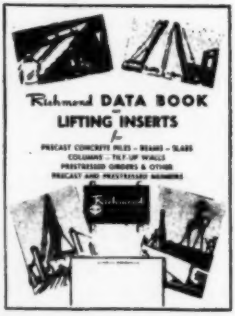

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Positions and Projects

Two California chapters hear speakers on steel reinforcing

Both of the California chapters of ACI held well attended May meetings featuring as dinner guest speakers well-known representatives of the reinforcing steel industry. The Southern California Chapter meeting was held in Los Angeles on May 18; the Northern California Chapter held a joint meeting on May 20 with Western Reinforcing Steel Fabricators Association.

In the informative talks presented, well illustrated by slides and film, emphasis was placed on the transition from old to new forms and practices, the technical advances made in recent years, and improvement in concrete design and construction employing both wire fabric and bar reinforcing.

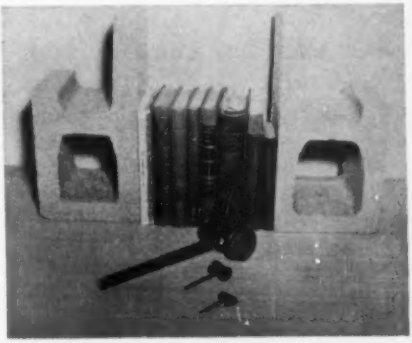
Frank B. Brown, managing director, Wire Reinforcement Institute, Washington, D. C., discussed a great variety of uses of wire fabric in building construction, pointed out advantages in reinforced concrete design using high strength and large size wire in fabric now available, and advantages in building construction resulting from shop assembly and economical handling and placing processes on the job. He was assisted by Don Gehring, in charge of public relations for the Wire Reinforcement Institute, in showing and explaining by means of slides and film strip, the procedures employed and the results obtained.

James T. Gearhart, assistant manager, reinforcing bar sales, Bethlehem Steel Co., Bethlehem, Pa., discussed the transition that has taken place in recent years in bar production and use, the trend toward the use of the new large size bars and the economies in concrete section dimensions which they make possible. He discussed welding practice, code provisions and limitations, compared ultimate strength and conventional design for typical situations, and thoroughly covered, with slide illustrations, the technical and practical aspects of bar reinforcing in concrete.

James T. Gearhart

Frank B. Brown

At the northern California meeting, a novel gavel made of reinforcing bars was presented to Harry E. Thomas, chapter president, by Robert H. Klinger, president of the Central Division of the Western Reinforcing Steel Fabricators Association. Realizing the strain of using such a heavy gavel, Rolf T. Retz, California State Division of Architecture, Sacramento, presented a miniature wooden gavel. Toward the end of the meeting, Otis D. Small, Palo Alto,



Novel gavel of reinforcing bars, presented in a spirit of good fellowship, is pictured with standard and miniature gavels also presented during the joint meeting

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presented an even smaller gavel, all three being shown in the photo.

Mr. Thomas remarked that as the bond between concrete and steel is being constantly proven, so may the bond of friendship and cooperation exist between the two organizations, for which an annual meeting is planned.

ACI President Joe Kelly, professor of civil engineering, University of California, Berkeley, and ACI Vice-President Lewis H. Tuthill, concrete engineer with the Division of Design and Construction, California State Department of Water Resources, Sacramento, both members of the section, were present.

Audiences at the two meetings, in which the reinforcing steel industry was well represented, totaled about 175.

ACI Michigan Chapter meets

The Michigan Chapter of ACI met for the usual monthly luncheon meeting on May 11, 1960.

Harvey Hanna was guest speaker for the meeting. Mr. Hanna, who is presently retained on a consultant basis by Hanna, Zabriskie and Daron, Inc., contractors of Detroit, was one of the founders and former president of the firm. He presented a talk on pneumatically applied mortar. Mr. Hanna covered the advantages and disadvantages of pneumatically applied mortar or pumped concrete, pitfalls to avoid, methods of control, quality of work, strength of the concrete, types of materials used, and a number of other factors pertaining primarily to this type of construction.

ACI headquarters attracts student touring groups

The Institute headquarters building has been included in the itinerary of a number of student tours passing through Detroit this spring.

The visitors' register shows: 40 students from the University of Illinois, Urbana; 32 students from University of Minnesota, Minneapolis; 30 architectural students from Virginia Polytechnic Institute, Blacksburg; 28 architectural students from Pennsylvania State University, University Park; 15 students from the University of Uruguay, Montevideo; and 10 associates from the Technical University, Braunschweig, Germany.

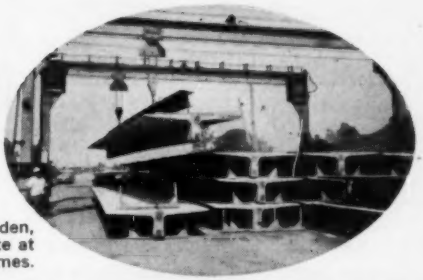
Pennsylvania State schedules two atomic shelter seminars

Pennsylvania State University, University Park, has organized an extensive program of instruction and research to acquaint architects and engineers with design and construction requirements for effective shelter against nuclear attack.

Two engineering seminars on atomic shelter are scheduled this summer. The first, July 10-22, is for architects and engineers who are or will become involved in planning and preliminary design aspects of buildings, shelters, and facilities to resist the effects of nuclear weapons. The second, July 24-August 5, will enroll structural and architectural engineers who are or will become involved in analysis and design of structural systems and radiation shielding systems of buildings, shelters, and facilities to resist the effects of nuclear weapons.

Gifford H. Albright, assistant professor of architectural engineering at Pennsylvania State University, directs the program, which is supported in

PRESTRESSED CONCRETE "T" BEAMS FOR UNIQUE PARKING DECK



Completed "T" beams at C. W. Blakeslee's Hamden, Conn. plant. They were shipped to the job site at the rate of 18 per day on trucks with special frames.

Placing beams at job site in Springfield, Mass. With each beam set at a slightly different angle, there is no more than a 4 percent grade in any part of the structure. Each beam is 8 ft. wide at the top of the "T" and only 8 in. wide at the stem.

• This new parking deck for the Forbes & Wallace department store is a striking example of the opportunities that prestressed concrete offers designers and builders.

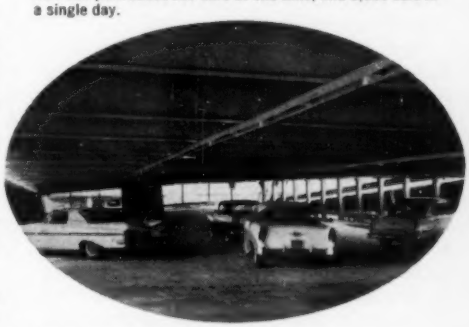
Gage and Martinson's unique design utilizes 182 ten-ton prestressed concrete "T" beams, whose 54′ lengths permit a completely post-free parking facility. Ease and speed of erecting the prefabricated concrete decks made it possible for the Ley Construction Company to open the 3½ story structure for business just seven months after breaking ground.

The manufacturer of these beams, C. W. Blakeslee & Sons, Inc. uses Lehigh Early Strength Cement for maximum production efficiency in their prestressing plant. Units are completed quickly, ready for trucking to the job site as needed.

This is typical of the advantages of Lehigh Early Strength Cement in modern concrete construction. Lehigh Portland Cement Company, Allentown, Pa.

LEHIGH CEMENTS

The 54′ spans provide ample room for angle parking on both sides of every level plus a wide center aisle for one-way traffic flow. Note complete absence of posts. The 3½ story structure can park about 350 cars at one time, and 2,000 cars in a single day.



OWNER: Forbes & Wallace, Springfield, Mass.

ENGINEER: Gage & Martinson, New York City

GENERAL CONTRACTOR: Ley Construction Co., Springfield, Mass.

MANUFACTURER OF PRESTRESSED BEAMS: C. W. Blakeslee & Sons, Inc., New Haven, Conn.

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part by the U. S. Department of Defense, U. S. Naval Civil Engineering Laboratory, and the Office of Civil and Defense Mobilization.

Professor Albright believes that architects and engineers can provide shelter against nuclear weapons in conventional buildings if they understand shelter requirements and the use of "convertible shelter." He points out that in every building program there are requirements for many functional spaces which lend themselves to use as effective shelter against nuclear attack if adequately planned, designed, and constructed with this objective.

The university has also instituted four graduate courses in nuclear defense planning and design and nuclear defense research.

Research will also be an important part of the program to be conducted at Penn State under the $117,000 initial grant provided by the three federal agencies.

Professor Albright points out that there are many things we still need to know. These range from radiation shielding and the response of structural systems subjected to blast loading to psychological, physiological, and sociological problems that may arise when when individuals spend prolonged periods in shelters. The program will include gathering and summarizing, as well as indexing, some of this information.

Professor Albright has participated in and directed research programs relating to shelter systems at Massachusetts Institute of Technology, U. S. Navy Bureau of Yards and Docks, U. S. Naval Civil Engineering Laboratory, Nevada Test Site, and Eniwetok Proving Ground.

Prestressed Concrete Institute convention

Program plans for the forthcoming 6th annual convention of the Prestressed Concrete Institute, September 27-30, at the Statler-Hilton Hotel in New York City, have been completed.

Utilizing the theme: "Prestressed Concrete, Key to Creative Architecture and Imaginative Engineering," the convention will be primarily a 3-day workshop designed to present new techniques and methods in this over $300 million industry.

The convention is open to all members of the institute, architects, engineers, manufacturers of material and equipment, and students. The program will include general sessions of interest to all attending plus special sessions directed primarily to producers and engineers.

Civil engineering education conference, July 6-8

A conference on civil engineering education will be held July 6-8 at the Rackham School of Graduate Studies on the University of Michigan campus, Ann Arbor.

This is the third and final conference in a series of three that have been planned with the support of the National Science Foundation and under the cosponsorship of Cooper Union, American Society for Engineering Education, and American Society of Civil Engineers.

The first two conferences, held at Green Engineering Camp of Cooper Union, were of a preliminary and planning nature leading up to this final conference to which delegates from all

colleges and universities having accredited civil engineering curricula are invited.

The objective of the series of conferences has been to evolve specific recommendations for the undergraduate curricula in civil engineering designed to meet current and future needs.

This third conference, involving about 150 engineering educators from schools of engineering in all sections of the country, will include presentations of concepts underlying trends to maintain the present curriculum or change the curriculum radically, discussion, and formulation of specific recommendations. A report of the conferences will be distributed to all departments of civil engineering and schools of engineering.

Mudd College inaugurates annual lecture series

The Harvey Mudd College, Claremont, Calif., inaugurated an annual lecture series on March 16. The series will be known as the Mendenhall Lectures, in honor of the sponsor, Irvan F. Mendenhall, president of Daniel, Mann, Johnson and Mendenhall, architects and engineers, Los Angeles.

Ernst Weber, president of Polytechnic Institute of Brooklyn, and president of the institute-owned Polytechnic Research and Development Corp., was first lecturer of the series. Dr. Weber spoke on "Preparing for the Unexpected," based on the problem of educating scientists and engineers for the unknown technology of the future.

Copeland honored by University of Wisconsin

R. E. Copeland, director of engineering, National Concrete Masonry Association, and a graduate of the University of Wisconsin, was one of six men widely known in engineering and industry who were honored at the 12th annual Wisconsin Engineers' Day celebration on the University of Wisconsin campus, Madison, in May.

Queen's University plans R/C design symposium

The Department of Civil Engineering at Queen's University, Kingston, Ont., Canada, has scheduled a symposium on reinforced concrete design, August 29 to September 2.

Three prominent ACI members, recognized authorities on structural engineering, headline the program: A. L. L. Baker, professor of concrete technology, Imperial College, London; Eivind Hognestad, manager, Structural Development Section, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.; and A. L. Parme, head, Advanced Engineering Group, PCA, Chicago.

The program is planned to give practicing engineers and others the opportunity to become familiar with recent developments in the field of ultimate load design and the design of shells.

S. D. Lash, head, Department of Civil Engineering, Queen's University, is general chairman of the symposium.

Hot Weather Concreting ■

RECOMMENDED PRACTICE FOR HOT WEATHER CONCRETING (ACI 605-59) spells out ways to minimize detrimental effects of hot weather on concrete. Means are described for reducing concrete temperatures by proper attention to ingredients, methods of production and delivery, and care in placement, protection, and curing. The use of admixtures to reduce mixing water requirements and to retard setting is also covered. 10 pp. Price $1.00, 50¢ to ACI members.

AMERICAN CONCRETE INSTITUTE **PUBLICATIONS**

International symposium on the chemistry of cement

The Fourth International Symposium on the Chemistry of Cement and Concrete will be held in the United States, October 2-7, with technical sessions at the National Bureau of Standards, Washington, D. C., beginning October 3. This will be followed by informal seminars on cement and concrete research to be held at the Research and Development Laboratories of the Portland Cement Association in Skokie, Ill., October 10-12.

The symposium in Washington will deal with seven topics: chemistry of clinker; chemistry of hydration of cement compounds; chemistry of hydration of portland cement; properties of cement paste and concrete; destructive processes in concrete; chemical additions and admixtures; and other hydraulic cements.

The meetings to be held at the PCA laboratories in Skokie are intended to acquaint visitors with Portland Cement Association research on cement and concrete and to permit their participation in informal seminars on cement manufacture and on basic and applied research.

Attendance at the symposium, which is jointly sponsored by the National Bureau of Standards and the Portland Cement Association, is by invitation. The official language of the symposium will be English.

Nebraska Prestressed awarded large piling, beam job

A large-volume contract for prestressed piling and beams was recently awarded to Nebraska Prestressed Concrete Co., Lincoln. Massman Construction Co., Kansas City, Mo., has ordered 31,000 ft (lineal) of octagonal-section prestressed concrete bearing piling. These members, ranging from 60 to 95 ft in length, are 14 in. in diameter.

In addition, the contract calls for 168 prestressed concrete beams, each 110 ft long and weighing 56 tons. The order is for Interstate Contract I-80-9(44), designed by the Nebraska State Highway Department in Lincoln.

Errata

The following corrections should be made in "Behavior and Strength in Shear of Beams and Frames without Web Reinforcement," by Roger Diaz de Cossio and Chester P. Siess, which was published in the February, 1960, ACI JOURNAL.

p. 699 — In the continued portion of Table 1, in the caption inserted at the head of the tabular data, the third line in this caption which reads: "Reinforcement: 3—# 4 bars; $p = 1.01$ percent" should read:

"Reinforcement: 2—# 9 bars; $p = 3.36$ percent"

* * * *

The following corrections should be made in "Elastic Analysis of Shear Walls in Tall Buildings," by Emilio Rosenblueth and Ignacio Holtz, which appeared in the June, 1960, ACI JOURNAL.

p. 1213 — in Eq. (2) the denominator is given as $h\phi m$; it should read $h\phi_m$.

p. 1221 — in Table 2 the last two column heads at the right side of the table are given as V_{w1} and M_{s1}; these should read V_{w1} and M_{s1}.

* * * *

The following corrections should be made in the discussion (Disc. 56-22) of "Comparison of Measured and Calculated Stiffnesses for Beams Reinforced in Tension Only," by Bill G. Eppes, which appeared in the June, 1960, ACI JOURNAL.

p. 1353 — the first full line directly under Table F that reads: "The stiffness of cracked section is almost proportional to percentage of steel" — is actually the heading for Table F.

Also on p. 1353 in the second sentence of the second paragraph under the italicized heading *First Example*, change that portion of the sentence which reads "... the curve near A is antimetric." to "... the curve near C is antimetric."

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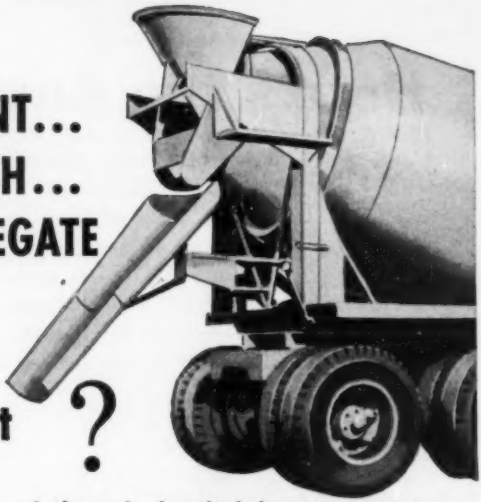

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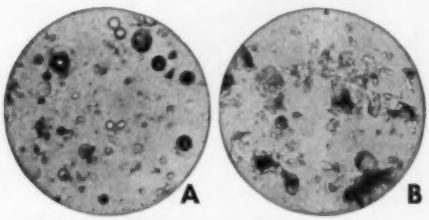

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ASEE elects 1960 officers

Eric A. Walker, president of Pennsylvania State Univeristy, University Park, is the newly elected president of the American Society for Engineering Education.

Melvin R. Lohmann, Oklahoma State University, Stillwater, and Newman A. Hall, Yale University, New Haven, Conn., have been elected vice-presidents. Wendel W. Burton, Minnesota Mining and Manufacturing Co., St. Paul, has been re-elected treasurer.

Hinds retires from Bethlehem Steel

Chester M. Hinds, Boston district engineer, Bethlehem Steel Co., retired. Mr. Hinds became Boston district engineer for Bethlehem Steel in 1936 and has been successively sales engineer, chief engineer, and since February, 1955, district engineer.

Master Builders names Pittsburgh branch manager

Andrew T. Fertal has been named manager of the Pittsburgh branch office of The Master Builders Co., Cleveland.

Mr. Fertal originally joined the organization in 1950 as a sales engineer. From 1952-53 he served with the Corps of Engineers as a construction engineer in Okinawa and as officer in charge of concrete construction. From 1953-59 he was associated with concrete and aggregate producers. In 1959 Mr. Fertal returned to the Master Builders Co. as sales engineer in the Pittsburgh branch office.

Howe joins Sika technical staff

Arthur K. Howe, a civil engineer experienced in construction and maintenance, has joined the technical service department of Sika Chemical Corp., Passaic, N.J. For the past 26 years he has engaged in engineering work dealing with construction, maintenance, and inspection of bridges, foundations, concrete structures and buildings. He has been associated with the Pennsylvania and Long Island railroads, the Air Force, the Nassau Bridge Authority, and a number of prominent engineering firms.

54 bridges in new Steinman contract

The State of Connecticut has retained D. B. Steinman, consulting engineer, New York, for the survey and design of an 8-mile stretch of new express highway through Cheshire, Southington, and Meriden. The contract requires a total of 54 bridges, including eight stream crossings, 42 grade separation structures, and four pedestrian overpasses.

In the town of Meriden, the route winds through the hills of Hubbard Park, where a wide median of variable width will permit each roadway to follow the contours without marring the beauty of the park. Structures in this area will be given special architectural treatment to harmonize with the surroundings.

The plans call for a limited access, four-lane divided highway with provision for future widening to six lanes.

Director post of CCAA retained by a Cavanagh

K. J. Cavanagh has succeeded his father, T. J. Cavanagh, as director of the Cement and Concrete Association of Australia, Sydney.

T. J. Cavanagh joined the association in 1936 as editor of *Constructional Review* and was appointed chief executive officer in 1941, assuming the position of director in 1945, which he held until his retirement last year.

K. J. Cavanagh joined the association in 1954 as materials engineer and was appointed deputy director in 1957, advancing to the director's post on his father's retirement.

AWWA names Friel "Water Man of the Year"

Francis Friel, Albright and Friel, Inc., Philadelphia, was presented the Fuller "Water Man of the Year" award for Pennsylvania at the 80th annual conference of the American Water Works Association held in Bal Harbour, Fla., in May.

Mr. Friel was cited in recognition of the eminence he has achieved in the field of engineering and the outstanding service he has rendered to the water utility profession and to the Pennsylvania section of AWWA.

Solite announces 1960 student award winners

Five senior students of architecture at the University of Virginia, Charlottesville, have won cash prizes totaling $750 in the seventh annual Solite Competition Award contest.

John Hoult Verkerke, Charlottesville, and William M. Walsh, Jr., Virginia Beach, tied for first place in the competition. Edward L. Gray, Jr., Baltimore, Md., and Henry G. Johnson, Jr., Hampton, Va., were second place winners. William W. Wood, Jr., Sands Point, N. Y., took third place.

Fourth year students of architecture at the university competed in the design of an exhibition building for Waynesboro, Va. The building is to be used as a display center for the arts, crafts, and industries of the Shenandoah Valley.

The competition is sponsored annually by Solite Corp., Richmond, Va.

Master Builders appoints three branch managers

The Master Builders Co., a division of American-Marietta Co., Cleveland, has named three branch managers as part of its expansion program in the southeast.

H. Jack Nunn has been named manager of the Atlanta branch office; H. J. McDargh, III, manager of the new branch at Jacksonville, Fla.; and Jack S. Peters, Tampa branch office manager.

Wheeler elected ABPA president

John Wheeler, Ontario Building Materials, Ltd., Toronto, Ont., Canada, was named president of the Autoclave Building Products Association at the 54th annual convention held at the Dinkler-Plaza Hotel, Atlanta, Ga.

Peter Smith of Edgar D. Otto and Son, Albuquerque, N. M., is vice-president of ABPA. Ralph Cromis, Boice Builders Supply Co., Pontiac, Mich., remains secretary and treasurer. John Selden, Toledo, continues as technical director.

In addition to the association's officers, the board of directors includes Dale Cobb, Jackson Ready-Mix Concrete Co., Jackson, Miss.; Fred Reinhold, Anchor Concrete Products, Buffalo; and Walter Horn, Cinder Block Inc., Detroit.

Construction to start on Foss Aqueduct

The contractor awarded the contract for the 51-mile long Foss Aqueduct, a major feature of the Bureau of Reclamation's Washita Basin Project in western Oklahoma, will be allowed 800 days to complete construction.

Features of the work include furnishing and laying 37½ miles of 42-, 30-, 27-, 21-, and 18-in. concrete pressure pipe; furnishing and laying 11.9 miles of 30-, 27-, 21-, and 18-in. pretensioned concrete pipe, or alternate cylinder prestressed concrete pipe, or alternate noncylinder prestressed concrete pipe; furnishing and laying 1.2 miles of 4-in. steel pipe, or alternate asbestos-cement or alternate cast iron pipe; and furnishing and laying 0.2 miles of 10-in. steel pipe, or alternate asbestos-cement pipe, or alternate cast iron pipe.

Major quantities for construction will include 408,000 cu yd of excavation, 115,000 lb of reinforcing steel,

46,000 lb of miscellaneous metalwork, 2880 bbl of cement, 45,000 lb of cast iron pipe and fittings, 30,000 lb of steel pipe specials, 24,000 lb of steel sleeves, nozzles, and covers, and 27,000 lb of steel pipe and fittings. Also to be constructed are eight railroad crossings and five highway crossings.

Elstner becomes associate of Engineers Collaborative

Richard Elstner, formerly structural development engineer, Portland Cement Association Research and Development Laboratories, Skokie, Ill., has become an associate member of The Engineers Collaborative, Chicago.

Active in Institute committee work, he is currently a member of ACI Committee 314, Rigid Frames for Buildings and Bridges, ACI-ASCE Committee 326, Shear and Diagonal Tension, and ACI Committee 339, Allowable Stresses in Reinforcement.

Southwestern Portland reports 1000-day safety record

Southwestern Portland Cement Co., El Paso, Tex., reports the enviable goal of 1000 days without a lost-time accident.

This safety record is a highly commendable accomplishment for a company the size of Southwestern Portland according to the National Safety Council. The Council reports that if the company's safety record had conformed to the nation's industry-wide figure, then its employees would have suffered about 20 lost-time accidents during this period.

Even more outstanding is Southwestern Portland's departmental safety record. Three of the firm's operating departments have unblemished safety records exceeding 30 years.

Management spokesmen for the company contend that the outstanding record is due to a broad program of safety

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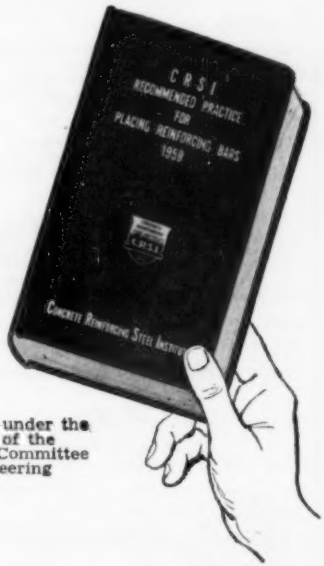

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- **Magnetite Iron Ore Concrete for Nuclear Shielding**—Physical properties and costs of magnetite ore concrete are compared to those of other concretes.
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education coupled with 100 percent cooperation on the part of the employees.

Recently Southwestern Portland received a first-place safety award from the Texas Safety Association for the second consecutive year for the cement industry in Texas. Also, the company has received for the second year running an award from the Portland Cement Association for going through a complete calendar year without a lost-time accident.

The National Safety Council has informed the company that it is eligible to receive a special award for surpassing 2 complete years without a lost-time accident.

Steinman awarded 25th honorary degree

David B. Steinman, internationally eminent bridge engineer of New York City, was recently awarded his 25th honorary degree.

The honorary degree of Doctor of Fine Arts was conferred upon Dr. Steinman at the formal convocation in May of the Spring Arts Festival at Bethany College in West Virginia. The degree is in recognition of his achievements as a poet and as a builder of beautiful bridges. Nine of Dr. Steinman's bridges have been honored in the annual awards for the most beautiful bridges in America.

Port Inland quarry awarded safety trophy

The Port Inland quarry operated by Inland Lime and Stone Co. Division of Inland Steel Co., Gulliver, Mich., was among six mines and quarries in Michigan, Minnesota, New York, and Pennsylvania awarded safety trophies in the 35th National Safety Competition sponsored by the Bureau of Mines, Department of the Interior.

The Port Inland quarry has operated 451,317 man-hr without a disabling injury. This is the fourth time Port Inland

has won the trophy. A participant in 27 successive contests, it ranked first in 1938, 1939, and 1951, and received honorable mention in 1933 and 1934.

Air Force okays use of prestressed concrete inlay

The Air Force is planning to permit use of a prestressed concrete inlay for the reconstruction of the primary runway and taxiway in Fairchild Air Force Base, Washington. This will be the first time in the United States that a prestressed concrete pavement project of such magnitude will be bid competitively with conventional concrete pavement.

The pavement scope for Fairchild AFB will be approximately 79,000 sq yd for the runway and 53,100 sq yd for the taxiways. Construction at Fairchild is tentatively scheduled to begin in the spring of 1961.

Southern Lightweight Aggregate appoints Laws to key sales post

Charles B. Laws was recently appointed southern regional sales manager for Southern Lightweight Aggregate Corp., Richmond, Va. He was formerly district sales manager for Carolina Solite Corp., a wholly owned subsidiary.

Flexforms affiliates with Prescon Corp.

Flexforms, Inc., originators of Flex-Tee, a prestressed single stem, pitched roof deck member, has become affiliated with the Prescon Corp., Corpus Christi, Tex.

The Flex-Tee roof deck member with a pitch of 1:12 has a pleasing appearance, according to Guy Braselton, president of Prescon, and is easily adapted to the various span requirements in the field so that it has wide application in many structures. It is available in a standard width of 6 ft with spans varying from 24 to 60 ft and can be cast either in the yard or at the job site.

1961 FIP-RILEM symposium on grouting for prestressed concrete

A symposium will be sponsored jointly by the Federation Internationale de la Precontrainte (FIP) and Reunion Internationale des Laboratories d'Essais et de Recherches sur les Materiaux et les Construction (RILEM) on injection grout for prestressed concrete. This symposium will be held at Norges Tekniske Hogskole in Trondheim, Norway, Jan. 5-7, 1961.

The technical program will cover general problems connected with the grouting of post-tensioned structures, composition of grout and methods of grouting, and standard requirements for grouting.

Sessions devoted to the results of laboratory and field studies will cover such properties as: fluidity, setting time, stability, shrinkage, frost resistance, and bond and other mechanical properties.

An exhibition and demonstration of materials and equipments for grouting will round out the program.

Papers intended for presentation at the symposium must be sent to Prof. I. Lyse, Norges Tekniske Hogskole, Trondheim, Norway, before Aug. 15, 1960. The FIP-RILEM Committee on Grout will decide which papers shall be presented at the symposium.

Conway joins Baltimore firm

George O. Conway, Jr., is now associated with National Paving and Contracting Co., Baltimore, as office designer, estimator, and job supervisor.

Columbia Cement appointments

A series of appointments in the sales organization of Columbia Cement Corp., Zanesville, Ohio, has been announced.

John R. Pinnick, Jr., has been named assistant sales manager in charge of highway sales; John K. Brown, district sales manager for the Columbus office; Eugene P. O'Brien, district sales man-

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ager at Columbia's new district office in Cleveland; and Clement J. Mertes, assistant sales manager for the Cleveland district, continuing to maintain his headquarters in Massillon, Ohio.

Columbia Cement, a wholly-owned subsidiary of Columbia-Southern Corp., will transfer its general sales offices from Zanesville to Columbus. The firm will continue to maintain order departments at both the Zanesville and Barberton plants to expedite customer service.

Hummel opens consulting office

John W. Hummel is a partner in the newly organized consulting engineering firm of Hummel and George, Louisville, Kentucky.

Watson joins Prescon Caribe

J. Raymond Watson, formerly instructor of civil engineering, University of Puerto Rico, Mayaguez, is now chief engineer of Prescon Caribe, Inc., Mayaguez, Puerto Rico.

Raj and Vakil open Bombay consulting office

Mahendra Raj and Rasvihari Vakil have announced the formation of their firm of consulting engineers under the name of Raj and Vakil in Bombay, India.

Mr. Raj came to the United States when he was an executive engineer in the Punjab Public Works Department and earned his MS at the University of Minnesota and CE degree at Columbia University. He worked for Ammann and Whitney, New York, for about 3 years.

Dr. Vakil obtained his engineering degree in India and his ME and PhD degrees at the University of Wisconsin. He worked with Ammann and Whitney for 2½ years, and taught at the University of Wisconsin, Columbia University, and Pratt Institute.

Executive changes at American Cement

Garner A. Beckett, formerly chairman of the Board of Directors of American Cement Corp., Philadelphia, has been elected honorary chairman of the board and Allen L. Chickering, Jr. has been elected chairman of the board, succeeding Mr. Beckett.

James P. Giles, Jr., president of Hercules Cement Co., a division of American, has been elected executive vice-president of American.

Alpha announces 6-year modernization plan

Alpha Portland Cement Co., Easton, Pa., will spend an estimated $50 million during the next 6 years to completely modernize all of the company's eight plants.

The modernization program is intended to help Alpha meet an anticipated boom in the construction field. Plans call for the expenditure of $6 million in 1960. The company has recently completed a $26 million expansion program.

Calaveras Cement opens Oregon transfer plant

Calaveras Cement Co., San Francisco, a division of The Flinkkote Co., has opened a new bulk transfer plant in Springfield, Ore. Charles Evans, Jr. has been appointed plant manager.

The plant will receive bulk cement by rail from Calaveras manufacturing plant at San Andreas, Calif., and serve as a distributing point for truck delivery in the 13 southern counties of Oregon.

Modern throughout, it can load one cement truck and trailer every 7 min. Its two storage silos have a total capacity of 2350 bbl of bulk cement. An adjoining warehouse holds four railroad cars of sack cement.

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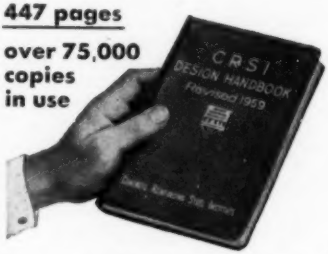

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Jan. 1 — May 31, 1960

Have you signed up a new ACI member this month? Among your acquaintances working in concrete, there are probably several excellent ACI prospects who would be ACI members if they only knew more about Institute activities. Tell them about ACI today and put your name on next month's honor roll.

Alfonso Marin E. 33
Samuel Hobbs 32
Harry Ellsberg 15
Walter H. Price 12
Joaquin Spinel L. 12

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1 point for Student; 2 points for Junior; 3 points for Individual; 4 points for Corporation; and 5 points for Contributing.

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Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. All who have an interest in concrete are eligible for membership. The grades of membership are described below.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI Journal provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings, they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

ACI's world-wide membership is growing in extent and participation —traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

Individual Members (U.S. and Possessions, Canada, Mexico, Central America, and West Indies)	**$20.00**
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Please enclose remittance with application (cut here)

Board of Direction, American Concrete Institute **Date**_____
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The undersigned hereby applies for admission to the American Concrete Institute as ☐ *Individual* ☐ *Corporation* ☐ *Contributing* ☐ *Junior* ☐ *Student Member and agrees to be governed by the Charter and Bylaws.*

Name and complete mail address of proposed membership (Address to which Journal is to be mailed—please letter) _____

_____

For Corporation Membership, ACI representative will be _____

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For our records, please complete both sides of application.

NOTES on Membership Classification

The ACI advertising and editorial departments need definite information concerning job title, classification, business affiliation, and principal responsibility of all members. Thus, the information requested on the classification form below is especially important.

The applicant should designate his title or position such as: construction superintendent, research director, technician, draftsman, structural engineer, inspector, plant superintendent, highway engineer. The occupation of the applicant should be that which is **most frequently** performed. The principal responsibility of the applicant is that specific area of his job for which he is **primarily concerned.**

Classification is not based on what interests you, but on what you do—what office or job you fill. When completing the form below, please check the one item in each section that is most applicable to that particular section.

ACI editors want to know your interests—they welcome suggestions as to what you'd like to see discussed in the ACI Journal and of the possible sources. Please attach a separate note or letter.

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☐ Arch ☐ Engr ☐ Construction Supervision ☐ Plant Management or Supervision ☐ Teaching ☐ Student ☐ Other (please state) .....

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New Members

The Board of Direction approved applications in the following categories: 67 Individual, 2 Corporation, 7 Junior, and 23 Student, making a total of 99 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership now stands at 10,274.

INDIVIDUAL

ALIBRANDI, JOHN G., Syracuse 6. N. Y. (Pres., J. G. A. Constr. Corp.)
ANTIA, KHURSHED A., Bombay, India (Part., Antia & Irani)
ARYA, ANAND SWARUP, Champaign, Ill. (Instr., Roorkee Univ. (India)
BAILEY, JAMES C., Atlanta, Ga. (Treas. & Prod. Engr., Conc. Mfg. Co.)
BENSON, F. R. Birmingham, England (Chf. Engr., British Lift Slab, Ltd.)
BLANKENSHIP, R. C., St. Louis, Mo. (Cons. Engr., PCA)
BURRINGTON, J., Chippenham, Wilts, England (Struct. Engr., Clarke, Nicholls & Marcel)
BUTLER, BRIAN G., Peterborough, Ont., Canada (Proj. Engr., Can. Gen. Elect. Co. Ltd.)
CAMPBELL, RICHARD W., Studio City, Calif. (Struct. Engr.)
CERULLI, ALEXANDER, Hempstead, N. Y. (Constr. Supv.)
CHAMBERS, STEPHEN, New York, N. Y. (Vice-Pres., City Investing Co.)
DARTNELL, GEORGE M., Teheran, Iran (Proj. Engr., IBEC Housing Corp.)
DECASTRO, J. L., Pinar del Rio, Cuba (Chf. Chem., Lone Star Cement Co.)
DOVE, ALLAN B., Hamilton, Ont., Canada (Prod. Dev. Mtlurgist, Steel Co. of Canada, Ltd.)
EBY, ROBERT W., New Albany, Ind. (Arch., Struct. Engr., Walker, Applegate, Oakes & Ritz, Inc.)
ELLSWORTH, RICHARD D., Pittsburgh, Pa. (Vice-Pres., Chas. F. Campbell, Inc.)
ESPING JOHN F., Kansas City, Mo. (Dist. Sales Mgr., Sika Chem. Corp.)
FAZAKERLY, GEORGE M., Memphis, Tenn. (Gen. Mgr., Fischer Lime & Cement Co., Inc.)
FULLERTON JOHN H., Reading, Mass. (Supv., Jackson & Moreland, Inc.)
GEARHART, JAMES T., Bethlehem, Pa. (Asst. Mgr. of Sales, Bethlehem Steel Co.)
GOLDSTEIN, LEO, Philadelphia, Pa. (P. E., Philadelphia, City of)
HAMADEH, SABKI M., Fresno, Calif. (Struct. Desr.)
HARDING, WENDELL, San Gabriel, Calif. (Conc. Techn., Wn. Conc. & Equip. Co.)
HARGRAVE-THOMAS, J. N., Detroit, Mich. (Gen. Mgr., Mich. Testing Engrs., Inc.)
HOOD, DONALD E., Virginia Beach, Va. (Supt., McLean Contr. Co.)
HOROWITZ, HAROLD, Richmond, Va. (Struct. Engr., Carneal & Johnston)
HURST, W. W., Lake Charles, La. (Plant Chf. Chem., Lone Star Cement Corp.)
JAEGERMANN, CHANOCH HEINZ, Haifa, Israel (C. E., Head, Testing Dept., Technion)
JARAMILLO, MANUEL F., Manila, Philippines (Struct. Engr., J. M. Zarogoza & Assocs.)
JOHNSON, M. W., St. Louis, Mo. (Mgr., St. Louis Dist., Pittsburgh Testing Lab.)
KAPLAN, STANLEY, Johannesburg, South Africa (Cons. Struct. Engr.)
KARKKAINEN, ARVID R., Rochester, N. Y. (City Engr., Rochester, City of)
KOWALSKI, T. G., N'dola, Northern Rhodesia (Struct. Engr.)

KUZARA, JOSEPH, Glasgow, Mont. (Supv. Engr. Hoiland & Lund, Archs.)
LAMOND JOSEPH F., Newton, Mass. (Matls. Engr., U. S. Army)
MANNERS, DEREK WILLIAM, Brighton, Sussex, England (Struct. Des. Engr., Chamberlain & Part., Cons. Engrs.)
MARK, NORMAN, Ashdod, Israel (Asst. Chf. Engr., Israel Off., F. R. Harris, Inc.)
MARSON, WILLIAM A., Santa Fe, N. M. (Brdg. Desr., N. M. State Hwy. Dept.)
MATRAJT, EDUARDO, Houston, Texas (Struct. Engr., Fish Engrg. Corp.)
McDANIEL, LEONARD F., Phoenix, Ariz. (Part., McDaniel & Powell)
MORELLI, J. F., Montreal, Que., Canada (Cons. Engr., Monarque & Morelli)
MOSBORG, ROBERT J., Urbana, Ill. (Assoc. Prof., Univ. of Ill.)
NORTHRUP, DONALD O., Pittsburgh, Pa. (Mgr., Constr. Prod., U. S. Steel Corp.)
PADHI, A. C., Parlakimedi, Orissa, India (Exec. Struct. Engr., Orissa, Govt. of)
PAEZ, JUAN B., Bogota, Colombia (C. E.)
PAGE, PHILIP P., Summit, N. J. (Assoc., Seelye, Stevenson, Value & Knecht)
PAYNE, DARRELL PAUL, Millbrae, Calif. (C. E. & Arch. Desr.)
PERRINE, E. L., Evanston, Ill. (Lab. Mgr., Zonolite Co.)
RAY, GORDON K., Chicago, Ill. (Mgr., Paving Bur., PCA)
RIVERSO, RAPHAEL G., Scarsdale, N. Y. (Constr. Supv., Riverso Constr. Co., Inc.)
SAMUELSSON, ALF., Jarnbrott, Sweden (Asst. Instr., Chalmers Univ. of Tech.)
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STOUT, GEORGE A., FPO, San Francisco, Calif. (Supv., Gen. Engr., Corps of Engrs.)
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TRAYNOR, DUANE, Bismarck, N. D. (Mgr., Des. Engr., Central Supply Co.)
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WADLIN, GEORGE K., JR., Orono, Me. (Assoc. Prof., Univ. of Me.)
WENK, PAUL, St. Gallen, Switzerland (C. E., Ing. bureau A. Zahner)
WILLIS, SHELBY K., Salina, Kan. (Part. Bucher and Willis)
YASHIRO, HIDEO, Chiba City, Japan (Asst., Inst. of Indust. Science, Univ. of Tokyo)
YOUSSEF, KAMAL E., Pittsburgh, Pa. (Desr., Swindell Dressler Corp.)
ZAHARIN, SERGEI A., Daly City, Calif. (Arch., Engr., R. B. Lilies)
ZAHNER, AD., St. Gallen, Switzerland (C. E.)

CORPORATION

EBASCO SERVICES INC., New York, N. Y. (H. K. Fairbanks, Cons. C. E.)
GEORGE C. FREY READY-MIXED CONC. CO., INC., Lancaster, N. Y. (Herbert G. Frey, Vice-Pres.)

Are You Listed Correctly?

Your ACI Headquarters staff has started work on the 1961 Membership Directory.

This Directory will list your last name, your first and middle initials, mailing address, membership classification (individual, junior, student, corporation, contributing), the year you joined ACI, and your job title and company affiliation.

If you receive the JOURNAL regularly, we have your correct mailing address. **But is your job title and company affiliation as listed in the 1959 Directory still the same?**

Complete the coupon below (please print) and mail today to ACI Membership Division, P. O. Box 4754, Redford Station, Detroit 19, Michigan. **If we don't hear from you, we will assume your 1959 Directory listing is still current.**

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Mailing Address City & State, Province or Country

Job Title Company Affiliation

OLD ADDRESS:

Mailing Address City & State, Province or Country

Job Title Company Affiliation

JUNIOR

BROERING, ARTHUR J., Covington, Ky. (Asst. S-3, USMC)
BROWN, HILTON RUSSELL, Sydney, NSW, Australia (Instr., Univ. of NSW)
GNECCO-CALVO, FRANCISCO, Bogota, Colombia (C. E., Cundinamarca, Govt. of)
KAMINSKI, JAMES G., Pittsburgh, Pa. (Des. Drftmn., Houser & Carafas)
KAWAGUCHI, MAMORU, Chiba City, Japan (Rsch. Engr., Univ. of Tokyo)
TELLAM, JAMES H., Toledo, Ohio (Sales Engr., Peerless Cement Co.)
THATCHER, M. LLEWELLYN, Watertown, Mass. (Grad. Stu., MIT)

STUDENT

ALONZO-VILLATE, GUILLERMO, Bogota, Colombia (Nat. Univ.)
BERGMEYER, MORITZ O., West New York, N. J. (Univ. of Notre Dame)
BIRKELAND, CHRISTIAN J., Seattle, Wash. (Univ of Wash.)
BLAKE, B. CAMPBELL, Birmingham, Ala. (Purdue Univ.)
BRUNALLI, JOHN, Milldale, Conn. (Univ. of Notre Dame)
ECKLIN, ROBERT L., Drumore, Pa. (Chicago Tech. Coll.)
FEIBUSCH, ROBERT J., San Francisco, Calif. (Univ. of Calif.)
GREEN, MELVYN, Los Angeles, Calif. (Univ. of Ariz.)
HADDORFF, DENNIS, Milwaukee, Wis. (Milwaukee Inst. of Tech.)
HICKS, GARDNER W., JR., Cambridge, Mass. (M. I. T.)
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LOPEZ-RIVERA, CARLOS M., Cayey, P. R. (Univ. of Puerto Rico)
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TORRES-LOZANO, JORGE, Bogota, Colombia (Univ. Nacional)
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Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Corner former

A specially designed, premolded, plastic material that produces a smooth, uninterrupted, rounded 1-in. radius corner in casting concrete is available from Servicised Products Corp. It can be used on columns, pillars, piers, beams, panels and many other types of precast concrete work where a smooth, rounded corner is desired. It is reported to be quickly and easily installed on standard forms, is reusable, eliminates sharp edges, and enhances architectural concrete.

The top end of the flanged section is fastened securely to the edge grain of one side of the form and then pulled taut. The corner former is then tacked or stapled every 4 in. The next step is to place the other side of the form in position and fasten securely, using conventional methods. to form the corner joint. This automatically closes the corner former to the required 90-dag angle and securely locks both feathered edges to the form.—Servicised Products Corp., 6051 West 65th St., Chicago 39, Ill.

Curing compound

Dekote, a curing membrane developed by Tretol, cures, hardens, seals, and dustproofs, all in one application, according to company spokesman. The clear, ready-to-use, liquid solution is applied using spray, roller, or brush equipment.

Prime features of Dekote cited by Tretol engineers mentioned the fast drying properties and ability to retain over 95 percent moisture; ability to seal the pores of concrete floors against most acids, oils, greases, and other foreign materials; and the fact that the compound does not have to be removed before the application of secondary floor treatments. Dekote can also be used for sealing and dustproofing existing concrete floors. Dekote meets ASTM C 309-58T.—Tretol, Inc., 7252 W. 66th St., Chicago 38, Ill.

Epoxy spraying gun

Pennsalt Chemicals Corp. has added a low pressure, high volume air, epoxy spraying gun unit to its line of products and services. Called Quikspray, the gun sprays materials of trowel consistency on floors and walls three to ten times faster than conventional application methods according to Pennsalt reports. The low pressure and high volume of air principle makes it possible to handle a wide range of epoxy formulations with or without aggregate.

The equipment is portable and easily assembled. It has an output of up to 10 cu ft per min. Snap connections for standard-sized air hoses and interchangeable orifices with rubber inserts make maintenance and cleaning easy according to the company.

The standard unit includes two aluminum cone-shaped hoppers so that mixing and spraying can proceed without interruption. Each hopper contains 1½ gal. of material. Power for a rotary-type compressor is provided by a 2½-hp gasoline engine or a ½-hp electric motor.

The unit can be used for applying acoustical plasters, stucco, sand finish plasters, colored aluminum and all aluminum-filled roof coatings, epoxy-based materials, latex-based materials, filled and nonfilled epoxy resins. Corrosion Engineering Products Department, Pennsalt Chemicals Corp., Natrona, Pa.

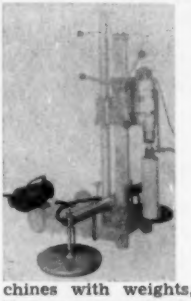
Stoop form panel

The Rocform Corp. has announced the availability of a stoop form panel accessory designed to be used primarily with the company's Rocform system, but it is so constructed that it may be used in most any forming system.

The form is made of 14-gage steel and ¾-in. plastic impregnated wood. It is tapered in two directions and held in place by removable clamps for easy stripping. It can be bulkheaded down to 18 in. from the top to accommodate any required grade. — Rocform Corp., 15160 W. Eight Mile Road, Detroit 35, Mich.

Vacuum hold-down attachment for drilling machines

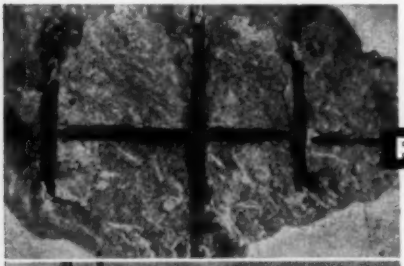
In just 10 sec the Grab-A-Slab vacuum attachment exerts 3000 lb of holding pressure, locking masonry drilling machines firmly in position against the surface to be drilled, according to the manufacturer. The unit fits all models of Kor-It machines, which use diamond core bits for high speed drilling of reinforced concrete and other masonry materials.



This attachment eliminates the need for positioning drilling machines with weights, bracing, or lag bolts. Two vacuum pads of special design grip rough surfaces as well as smooth.—The Kor-It Co., Inc., 991 Richard Ave., Santa Clara, Calif.

Gas-fired portable heaters

Portable gas-fired infra-red radiant heaters have been introduced by Perfection Industries. The unit can be operated outdoors or in using bottled propane gas.



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The heaters are available in three sizes, the smallest, a portable "Handi-Heater" weighs 8 lb and has a rated input of 16,000 Btu, operating at less than 5 cents per hour according to the manufacturer. Two larger sizes are mounted on a vertical support rod attached to a wheeled propane bottle carrier. The larger model is rated at 36,000 Btu input and the smaller unit has rated output of 24,000 Btu.—Perfection Industries, Division of Hupp Corp., 1135 Ivanhoe Rd., Cleveland 10, Ohio

Cork expansion joint

Cork expansion joints for concrete have been developed by National Expansion Joint Co. to meet the requirements of the bridge department of the State of California.

The product, called Naco cork expansion joint, will compress to one-half its thickness under a load to 50 psi. With release of the pressure, the expansion joint will return to within 95 percent of its original size, according to the company.

The cork is asphalt impregnated and encased within 15-lb felt. It is reported to weather well and to have superior handling qualities. Standard sizes are available.—National Expansion Joint Co., 1601 Embarcadero Road, Oakland, Calif.

Slab grinder

Equipment Development Co. has announced the addition of a new model to its line of concrete slab grinders, the Model JR, designed for light concrete grinding.

Model JR can be used for grinding dry concrete slab, rain pitted slab, joints, high spots, and ridges. It will also remove trowel marks, and surplus concrete and plaster.

This model is powered by a ¾-hp, 110/220 volt ac motor. Counter-rotating twin discs in neoprene mountings provide torque-free operation. The unit weighs less than 100 lb and has a grinding area of 112 sq in. The over-all height of this model is 37 in. with a width of 15 in. and an over-all length of 19½ in. It employs six 1 x 2 x 3 grinding stones.—Equipment Development Co., 2700 Garfield Ave., Silver Springs, Md.

Pavement coring machine

The Mobile Mark V pavement coring machine is designed for production test coring on highways, aircraft runways, and other slab structures. Because of its lightweight, one-piece design, the unit may be mounted on or demounted from any ¼-ton truck or trailer in a matter of minutes according to Mobile spokesman.

The hydraulic drill motor accommodates diamond bits from 2 to 10 in. in diameter. The self-powered, self-contained Mark V has a three-way hydraulic drive that permits coring through steel reinforced concrete at the rate of 30 sec per in. An hydraulic control automatically maintains the correct drilling pressure and rate of feed and automatically

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Design of Concrete Pavements

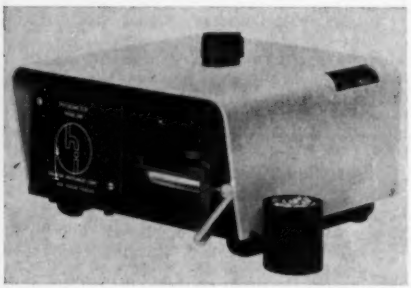
Recommended Practice for Design of Concrete Pavements (ACI 325-58) covers the design of rigid airport and highway pavements and bases for conditions of climate, traffic, available construction materials and equipment, and construction methods of the United States. Includes recommendations for soil foundations, selection of slab dimensions, joints, and details for reinforced or nonreinforced concrete. 36 pp. $1.00 per copy, 50¢ to ACI members.

Order from Publications Department, American Concrete Institute, P. O. Box 4754, Redford Station, Detroit 19, Michigan.

compensates for minor changes in material hardness.—Mobile Drilling, Inc., 960 N. Pennsylvania St., Indianapolis 4, Ind.

Air comparison pycnometer

The Houston Model 200 air comparison pycnometer incorporates a unique principle for rapid nondestructive volume, density, and porosity measurements on irregular, pow-



dered, and porous solids, including concrete Other uses are: determining moisture content, void volume, and entrapped air determination.

Designed for laboratory, industrial, and field usage, the unit is completely hand operated and requires no power source. The unit requires no liquids, no gas pressure determination and no temperature control. Although air is used as the fluid, the Pycnometer's accuracy does not depend on the perfect gas laws.

It is said that a complete measurement may be made by an unskilled operator in 1 min without wetting or damaging the sample.—Houston Instrument Corp., P.O. Box 22234, Houston 27, Tex.

Pliable sealant

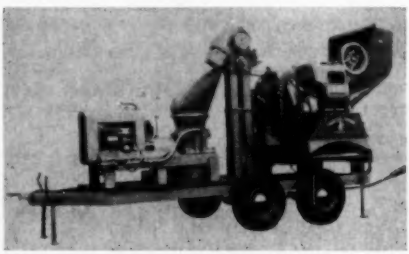
E-Bond No. 1007, formulated by International Epoxy, is a permanently pliable sealant compound based on Thiokol free flowing liquid polysulfide polymers. It cures to a tough rubber-like seal that will not shrink, crack, oxidize, or become brittle, according to the manufacturer.

Chemically inert it resists acids, alkalies, and solvents. It is recommended by the company for waterproofing the joints of curtain walls, windows, wall facings, flashings, expansion joints, or folded plate roofs.—Laboratory, International Epoxy Corp., 501 N. E. 33rd St., P.O. Box A-931, Fort Lauderdale, Florida.

Shotcreting equipment

To simplify transportation and reduce manpower requirements, Engineered Equipment, Inc., offers the Jetcreter mounted on its own trailer, complete with an 11S mixer as well as bucket elevator.

Mixers of various sizes and types and an alternate belt conveyor instead of bucket



elevator are available. With Super Jetcreter and 600 ft compressor, the unit will produce up to 12 cu yd per hr according to company spokesman. — Engineered Equipment, Inc., Waterloo, Iowa.

Materials handling

Vac-u-lift announces a self-contained lifting unit. Company reports it to be an economical and versatile single pad unit for safe, efficient handling of barrels, stone, concrete, sheet and plate steel, and other nonporous materials.

Admixtures for Concrete

• *By ACI Committee 212*

Concrete admixtures are classified into 11 groups: accelerators, retarders, air-entraining agents, gas-forming agents, cementitious materials, pozzolans, alkali-aggregate expansion inhibitors, damp-proofing and permeability-reducing agents, workability agents, grouting agents, and miscellaneous.

This 34-page report discusses each group and the important effects to be expected in using materials of each group.

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The standard unit is approximately 18 in. high and is equipped with a 16 in. diameter pad and neoprene seal ring with a lifting capacity of 1200 lb in horizontal lift and 600 lb lift in shear position. Pad Pac is self-contained and operated by a 1/3 hp Power-pac attached directly to the pad.—Vac-U-Lift Co., Division of The Siegler Corp., Salem, Ill.

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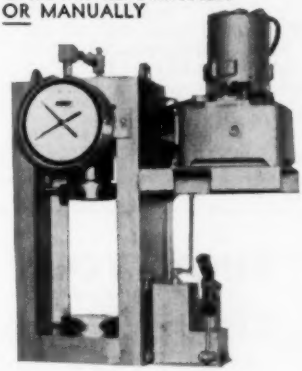

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Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

HEAVY-DUTY SCAFFOLD (Bulletin No. 73)—Four-page illustrated folder describes Advance 10-M heavy-duty scaffold, designed to support extra-heavy concrete slabs and beams, and particularly applicable for use in the shoring of bridges and other heavy spans. The accessory items for use with the scaffold are also illustrated and described.—Beaver-Advance Corp., P.O. Box 792, Ellwood City, Pa.

WATERTIGHT CONCRETE (Bulletin P-49b)—Summary of authoritative information on the design and specification of watertight concrete is given in this 6-page publication. Discussion covers the basic requirements for watertight concrete and how Pozzolith reduces shrinkage, bleeding, and segregation.—The Master Builders Co., Cleveland 3, Ohio

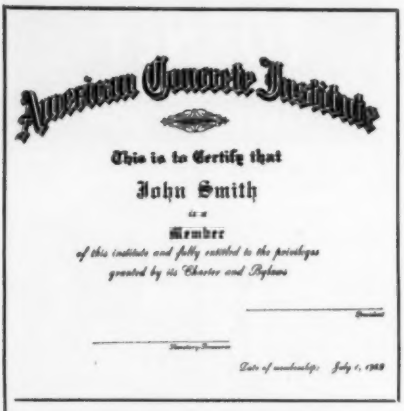
STRUCTURAL DESIGN EXAMINATION—NEW JERSEY—A set of solutions, compiled by Odd Albert, to the structural design section of the professional engineers examinations in New Jersey taken from examinations for 1957-59. 14 pp., mimeographed, $1 per copy.—Opla Co., Box 266, Belmar, N.J.

GRADUATE CE SCHOLARSHIPS—A survey of graduate scholarships, fellowships, and assistantships available to civil engineering students in 1960 has been prepared by Chi Epsilon, national civil engineering honor fraternity. The graduate programs include over 500 individual awards.—Michael A. Spronck, Chi Epsilon Fraternity, Martinsville, N.J.

SCREW CONVEYOR ENGINEERING AND SPECIFICATION GUIDE (Catalog Section 200-C)—Fort Worth Steel and Machinery has published a 60-page guide for screw conveyor installations to handle a wide variety of bulk materials, including cement. Included are complete listings of screw conveyor components and accessories, with sizes and dimensions. Lists approximately 300 different materials, classified by physical characteristics which affect conveyor selection.—Fort Worth Steel and Machinery, 3600 McCart St., Fort Worth 10, Tex.

WATER REDUCTION BULLETIN (Technical Paper No. 1)—Bulletin features a technical paper entitled "Reduction of Water in Concrete Mixes by Chemical Admixture; Effect on Properties," providing comprehensive technical information on the subject and on WRDA, Dewey and Almy water-reducing agent.— Dewey and Almy Chemical Division, W. R. Grace & Co., Cambridge 40, Mass.

PERMALITE INSULATING CONCRETE—Bulletin covers the use of Permalite expanded perlite insulating concrete in roof decks and floor fills. The 8-page bulletin includes tables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showing typical mix proportions, physical properties, and thermal conductivity of Permalite, as well as typical load test data for roof decks at various spans. Also included is an analysis of Permalite insulating concrete fire ratings. — Perlite Department of Great Lakes Carbon Corp., 612 S. Flower St., Los Angeles 17, Calif.

INTEGRALLY COLORED CONCRETE—Catalog covers product description, use, methods of application, and colors of Colorundum, a ready to use powder, which is dusted on and trowelled into freshly placed concrete to produce colorful concrete.—A. C. Horn Companies, 2133 85th St., North Bergen, N.J.

TESTLAB DIGEST (No. 2)—Publication contains 16 pages devoted to testing apparatus relating to concrete, soils, bituminous, and general testing.—Testlab Corp., 3398 Milwaukee Ave., Chicago 44, Ill.

PUBLICATIONS OF THE ENGINEERING AND INDUSTRIAL EXPERIMENT STATION, 1953-1959—Leaflet No. 116 of the Florida Engineering and Industrial Experiment Station, University of Florida, lists the publications issued by the station under the general title of "Engineering Progress at the University of Florida." There are five series: bulletin series, technical paper series, leaflet series, technical progress reports, and Florida engineering series. In addition to listing the publications in the various series there is also a subject and author index.—Florida Engineering and Industrial Experiment Station, College of Engineering, University of Florida, Gainesville, Fla.

HORNFLEX THIOKOL LP-32 SEALANT — 16-page catalog on Hornflex Thiokol LP-32 sealant for watertight joint and weld on curtain walls, prefabricated panels, glass enclosures, roofs, mullions, flashings, store fronts, sidewalks, coping, flooring, pipe joints and passages, air conditioning ducts and units. Describes physical properties, uses, advantages, and directions for surface preparation and application.—A. C. Horn Companies, 2133 85th St., North Bergen, N.J.

CEMENT INDUSTRY IN EUROPE — 1957 STATISTICS—Compilation of statistics on the cement industry in Europe by the Organisation for Européene de Coopération Economique. Compares 1957 results with previous years and outlines trend for 1958, giving some indication of investment programs through 1959. (32 pp., $1.00)—Organisation for Européene de Coopération Economique, Publications Office, 1346 Connecticut Ave.,

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This ACI Bibliography (No. 2 in the series) lists and annotates 35 selected articles appearing in available technical publications issued from 1924 to 1958 and dealing with compression tests of concrete, variations in test results, and evaluation of tests. The articles explain the advantages and value of applying statistical methods in evaluating quality of concrete. In 8½x11-in. format, punched for insertion in three-ring binder. Price per copy is $2.00; $1.00 to ACI members.

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